57AA

STATIC AND CYCLIC LATERAL LOAD TESTS ON INSTRUMENTED PILES IN SAND

by

Lino Cheang
and
Hudson Matlock

The Earth Technology Corporation
3777 Long Beach Blvd.
Long Beach, California 90807

October 1983

PREFACE

In July 1981, pile vibration tests sponsored by the National Science Foundation (NSF) were performed at Seal Beach, California. These vibration tests were carried out jointly by the Earth Technology Corporation (Ertec) and Professor Ronald F. Scott from the California Institute of Technology. Two piles were installed but only one was actually instrumented and used in the vibration testing.

The existence of these two piles provided an excellent opportunity to pursue further studies of pile behavior. Accordingly, a proposal to do additional lateral testing was issued in October 1981. The new program called for static and slow cyclic tests under a variety of pile-head restraints for purposes of comparison with current design criteria. This program was funded by eight oil companies and the Minerals Management Service. The project officially began in April 1982.

Following is a list of the sponsors of this project and their representatives:

Chevron Oil Field Research Company	Gordon E. Strickland Jen-Hwa Chen
Conoco Inc.	Jack H. C. Chan
Exxon Production Research Company	Dick Raines
Gulf Oil Exploration and Production Company	Steve K. Paulson

TABLE OF CONTENTS

		Page
PREFACE		i
EXECUTIVE SUMMARY		vi
1.0	INTRODUCTION	1-1
2.0	PRETEST ANALYSIS	
	2.1 Purpose	2-1
	2.2 Design and Construction of Test Pile and Loading System	2-1
	2.3 Sensitivity to Friction Angle	2-5
3.0	GEOTECHNICAL INVESTIGATIONS	
	3.1 Background	3-1
	3.2 Supplemental Site Investigation Program	3-2
	3.3 In Situ Testing	3-3
	3.4 Soil Sampling	3-4
	3.5 Summary of Laboratory Test Results	3-6
	3.6 Soil Parameters Used in Analysis of Pile Test Data	3-7
4.0	PHYSICAL TEST SYSTEM	
	4.1 General	4-1
	4.2 Load Frames and Hydraulics	4-1
	4.3 Instrumentation	4-3
	4.4 Data Acquisition	4-7
	4.5 Pile and Load Cell Calibrations	4-10

TABLE OF CONTENTS
(continued)

5.0 FIELD TESTS

5.1 General 5-1

5.2 Test Control. 5-1

5.3 Test Sequence 5-2

5.4 Loading Rate. 5-6

5.5 Observation of Soil Deformation Near Pile Surface 5-6

6.0 PRESENTATION OF TEST RESULTS

6.1 Measurements. 6-1

6.2 Pile Head Behavior. 6-1

6.3 Bending Moment Distribution 6-5

6.4 Pore Water Pressure Response 6-9

6.5 Pile Inclination Measurements 6-10

7.0 ANALYSIS OF TEST RESULTS

7.1 General 7-1

7.2 Analytical Tool 7-2

7.3 Detailed Analytical Procedure 7-4

7.4 Evaluation of API p-y Criteria 7-7

7.5 Soil Resistance Variation During Cyclic Loading 7-12

7.6 Comparison of Pile 1 and Pile 2 Behavior. . . . 7-14

7.7 Boundary Conditions 7-16

7.8 Discussion. 7-17

TABLE OF CONTENTS
(continued)

8.0 SUMMARY AND RECOMMENDATIONS

8.1 Summary 8-1

8.2 Recommendations 8-3

9.0 REFERENCES 9-1

APPENDIX. Locations and Results of Previous NSF Site Investigation Program

EXECUTIVE SUMMARY

Background

In the summer of 1981, two steel pipe piles were installed at Seal Beach, California. One of the piles was then instrumented and used in a series of vibration tests. These vibration tests, sponsored by the National Science Foundation (NSF), were performed jointly by the Earth Technology Corporation and Dr. Ronald F. Scott of the California Institute of Technology. The vibration tests were completed and a final report was submitted to NSF in December 1981 (Ertec, 1981; Scott et al, 1982). The existence of these piles provided an excellent opportunity to pursue further studies of pile behavior.

In October 1981, copies of a proposal were sent by the Earth Technology Corporation to several companies and government agencies to solicit support for a program of static and slow cyclic lateral load tests. The purpose of this study was to evaluate the validity of some aspects of current API recommended practice. This study was subsequently funded by eight oil companies and the Minerals Management Service and was initiated in April 1982.

Instrumentation and Site Investigation

The test piles were 24 inches (61 cm) in outside diameter with a uniform wall thickness of 0.5 inch (1.3 cm). The penetration was 32 feet (9.8 m). Both piles were instrumented

by strain gages to measure bending moment along portions of the pile length. Pile-head load and deflection were measured by strain-gaged load cells and LVDT's at two different elevations above the ground surface. Pore pressure measurements were also attempted and pile inclination was monitored periodically by a precise levelling device.

Two separate site investigation programs consisting of cone penetrometer testing (CPT) and soil sampling were conducted. The first geotechnical investigation was performed for the NSF vibration tests and the second supplemental investigation was done for this study. Results from the two investigations indicated the soil medium at the pile locations consisted mainly of medium dense silty sand. However, from the supplemental CPT results obtained closer to the piles, a silty clay layer was detected between the depths of about 5 and 8 feet (1.5 and 2.4 m) below the ground surface. This clay layer was not evident from the first CPT records taken 30 feet (9.1 m) away from the test piles. The presence of this clay layer provided an ideal opportunity to observe the effect of a layered soil system on lateral pile behavior. This is a matter of considerable general interest and speculation, but the condition did require some changes in the original purpose and scope of the study.

Pile Loading and Data Acquisition

The loading system consisted of a telescoping unit assembled from 5 and 6-inch (12.7 and 15.2 cm) heavy wall standard pipes. The telescoping unit was mounted between the test piles and supported near the ground surface. Four hydraulic rams, acting through the telescoping unit, were used to apply the lateral load at 1 foot (0.3 m) above the ground surface. Another 6-inch (15.2 cm) pipe was connected across the two pile tops. Part of this strut consisted of a threaded rod. Two large threaded bearing wheels were used to rotate nuts to control the pile top movement and thus the boundary restraint.

A total of ten tests were performed under three different sets of boundary conditions. The three boundary conditions are to a certain degree artificial, but they cover the full range of conventional pile-structure connections. The initial and the last tests were done under a condition where lateral pile deflection at the pile tops was adjusted progressively. These tests were designated as partially restrained-head (PRH). The second through sixth tests were cyclic free-head (FH) tests wherein the lateral movement of the pile tops was unrestricted. The seventh through nineth tests were termed fully restrained-head (FRH) tests. These tests were cyclic and the pile tops were not allowed to displace laterally. The test program consumed a total of three days beginning on 22 September 1982.

Digital data acquisition and storage were controlled by a microcomputer. Two analog plotters were used to display selected data in the field. All digital data were permanently stored on a built-in cassette recorder which was part of the microcomputer.

Data Analysis

Because of the limited number of bending moment measuring stations, there was no intention in this study to obtain p-y curves directly through mathematical differentiation and integration of the measured bending moment curves. Instead, it was planned to input p-y curves in a computer model to calculate bending moment and pile-head responses during lateral loading. The computer results were then used to compare with field measurements to determine the validity of these p-y curves.

The SPASM computer program was used in the analyses instead of the more common beam-column method because it provides for both nonlinear and inelastic soil supports. The program thus enables the modelling of both loading and unloading. The soil support model was defined by backbone (initial loading) curves which correspond to conventional p-y concepts. The curves are converted in the program to mechanical analogs which provide the soil response under any sequence of loading. Because of the presence of the clay layer, both the clay and sand criteria recommended by API were used to derive the p-y curves. Overburden pressures were accumulated to the depth of each p-y curve. Actual measured pile deflections were used to control the computer model.

Results of Analysis

The SPASM analyses indicated little difference in computed results between the static and cyclic API p-y curves for sands. The SPASM analyses together with the test data also suggest that the design of lateral piles is insensitive to variation in p-y curves. For the restrained-head cases, lateral pile design is dominated by the boundary restraint assigned in the analysis.

During the controlled-displacement cyclic loading tests, gradual reduction in soil resistance (degradation) was measured. Grain migration and soil compaction were also thought to have occurred during cyclic loading. Grain migration is the process by which soil moves down at the back of the pile during loading. In this test program, the results indicated that the effect of grain migration on lateral pile response was small.

The lower bound (equilibrated) pile-head load-deflection relationship obtained for the cyclic tests produced fairly good agreement when comparing the calculated and measured values. A similarly good agreement was obtained for the virgin static test when the static p-y curves were used in the analysis. Selected bending moment distributions were also used in the comparison. The agreement was again very good. These sets of comparisons resulted in the conclusion that the API methods of cyclic p-y curve formulation are applicable for lateral pile analysis in a layered soil system which is dominated by sand.

1.0 INTRODUCTION

Soil characterization for offshore lateral pile design for sand is based largely on the results of a single research program conducted by Reese and others (Reese et al, 1974; Cox et al, 1974). The data were obtained from free-head pile tests and were used to develop a procedure for constructing characteristic curves of soil resistance p versus pile deflection y for either static or cyclic loading. Since the criteria are based on one set of soil and pile parameters, the uncertainty of applicability to other sites and to other typical conditions of pile-head restraint provides strong justification for additional testing.

Under an earlier program sponsored by the National Science Foundation (NSF) two 24-inch (61 cm) diameter steel pipe piles with a wall thickness of 0.5 inch (1.3 cm) were installed at Seal Beach, California. The penetration depth was 32 feet (9.8 m) below the ground surface. In the NSF study, one of the piles was instrumented and used in vibration tests. These tests were performed jointly by the California Institute of Technology and the Earth Technology Corporation and were completed in July 1981 (Ertec, 1981; Scott et al, 1982). The availability of these piles provided an excellent opportunity to conduct some follow-up tests which are the primary subject of this report.

A static and cyclic lateral load test program was proposed by the Earth Technology Corporation in October, 1981, and was subsequently funded by eight oil companies and the Minerals Management Service (MMS) of the United States Department of the Interior. The work began in April, 1982.

The work began with the design of additional test arrangements so that lateral loads, both static and slow cyclic, could be applied to the piles. A loading system was developed to provide various combinations of free and restrained pile-head conditions. The scheme was checked by computer simulation of pile response. Instrumentation was installed to monitor the bending moment along both piles as well as the deflections and applied loads at the top. Transducers were installed to monitor pore pressures in the soil during pile loading, and pile-head inclination was measured periodically with a precise level.

The soil investigation program for the earlier NSF program had been done before the piles were installed. The piles were then driven, about 30 feet (9.1 m) from the nearest boring, at a location which would be covered by shallow water at the time of testing. An additional geotechnical investigation was performed for this study and consisted of piston sampling and cone soundings immediately adjacent to the piles. A 3-foot (0.9 m) thick silty clay layer, which was not apparent in the earlier investigation, was detected by these later cone soundings. The existence of this clay layer caused the direction of this study to be modified. This rather ideal layered soil system provided an opportunity to examine the applicability of the joint usage of both API sand and soft clay criteria for soil characterization, a question which has been frequently raised in recent years.

Because of the limited number of strain gage stations, p-y curves were not deduced by direct integration and differentiation of the measured bending moments as described by Matlock and Ripperger (1958). Instead, in analyzing results for this study, p-y curves obtained from API recommendations were used to calculate bending moment and load-deflection relationships which were then compared with the measured data. The comparisons provided a means for evaluating the API methods.

This report summarizes the pretest analysis, site investigation, field testing and results. The relevant field measurements are given together with the results of the test simulation using the computer program, SPASM. Conclusions and recommendations are also given at the end to summarize the principal findings.

2.0 PRETEST ANALYSIS

2.1 Purpose

One of the objectives of this test program is to study the behavior of laterally loaded piles with head restraints covering a range typical of real offshore piles. This requirement was considered in the design of the loading system and test piles. Since it would have been difficult and too restrictive to construct and install a form of superstructure that would directly simulate typical head restraints, an alternate procedure was used.

Before design of the system, an analysis was performed to calculate typically expected bending moment and shear forces at the mudline for an assumed prototype pile and jacket combination. The loading system and test piles were then designed to approximately duplicate the desired combination of the prototype shear and moment. A more detailed discussion of this procedure and the analytical results is given below.

2.2 Design and Construction of Test Pile and Loading System

As indicated earlier, two test piles originally had been installed for a series of vibration tests which were completed in July, 1981. The outside diameter of each pile was 24 inches (61 cm) and the wall thickness was 0.5 inch (1.3 cm). The total pile length was 40 feet (12.2 m) with a stick-up of 8 feet (2.4 m) above the ground surface. The center-to-center spacing of the two piles was 12 feet (3.7 m). A load frame was required to be designed and installed between the existing piles to provide the push-and-pull actions needed for static and cyclic loadings and a simulation of head-restraint.

The design requirements were accomplished by (1) a lateral analysis of the assumed prototype restrained-head pile to estimate shear and moment at the mudline and (2) development of a loading system for the test piles which would provide similar combinations of shear and moment.

Prototype Pile Analysis. The prototype pile assumed in this analysis had the same dimensions as the two test piles except that the height above ground was increased to 30 feet (9.1 m). Equal lateral deflections were input at one and 30 feet (0.3 and 9.1 m) above the mudline. The computer program BMCOL 76 (Matlock et al, 1981) was used. The pile and the model used in the analysis are shown in Fig. 2-1.

The prototype analyses were done before presence of the clay layer was known. The soil mass was therefore assumed to be a uniform deposit of fine sand characterized by a friction angle ranging from 25 to 35 degrees to accommodate possible soil variation. Sensitivity of pile solutions to variation of friction angles is addressed later in this chapter. Using the extreme friction angles, two sets of lateral soil support (p-y) curves were derived based on the recommendations given for sand in API RP 2A (1982) and were then incorporated in the BMCOL model.

Deflections at two movable supports were used to simulate the action of the prototype structure on the pile. The results of the analysis are presented in a summary plot of maximum negative bending moment and lateral force at the lower support versus imposed support displacements. These results, shown in Fig. 2-2, were used to guide the design of the test pile and loading system.

Test Pile Construction. It would have been impractical to test piles with a 30-foot (9.1 m) elevation above the ground surface; furthermore, it was desirable to have easy access to the top of the piles. An unused 10-foot (3 m) cut-off of the same test pile was available from the previous vibration tests, so each of the test piles was extended 5 feet (1.5 m), resulting in a total height above ground of 13 feet (4 m).

After the actual test pile height was selected, an analysis was performed to estimate the required load and deflection which would produce approximately the same boundary conditions as the prototype. The boundary conditions selected for the match were defined by a combination of shear and moment at the lower support of the prototype pile at a bending stress of 36 ksi (247,000 kPa). The lower support was located at 1 foot (0.3 m) above the ground surface. The controlling magnitudes of the prototype shear force and bending moment are taken from Fig. 2-2 for a friction angle of 35 degrees. The computer program BMCOL 76 was again used for the computation.

The problem description, which includes the test pile configuration and the prototype case for comparison, is shown in Fig. 2-3. To estimate the required capacity of the loading system, the upper-bound friction angle of 35 degrees was used in the analysis. The test loading condition was defined by a total of two lateral loads imposed at the pile top and at one foot (0.3 m) above the ground surface.

Magnitudes of the support movements were calculated by the BMCOL program. The results of the analysis are shown in Fig. 2-3(b). The imposed lateral load is -61 kips (271 kN) at the pile top and 197 kips (876 kN) at one foot (0.3 m) above ground surface. The corresponding deflections are 3.5 inches (8.9 cm) and 2.6 inches (6.6 cm). These magnitudes of load and deflection indicated that with the test piles extended to 13 feet (4 m) it was feasible to construct a practical loading system.

Loading System Construction. The remaining task of this design effort was to devise a loading system which would provide the required combinations of load and deflection at the two levels. To replace the two loads or supports in the computer model with a physical system, one load strut was required to control and limit the lateral deflection at the top of the test piles and another to apply the horizontal load at about one foot (0.3 m) above the ground surface. Detailed designs of these load struts are presented in Section 4.2.

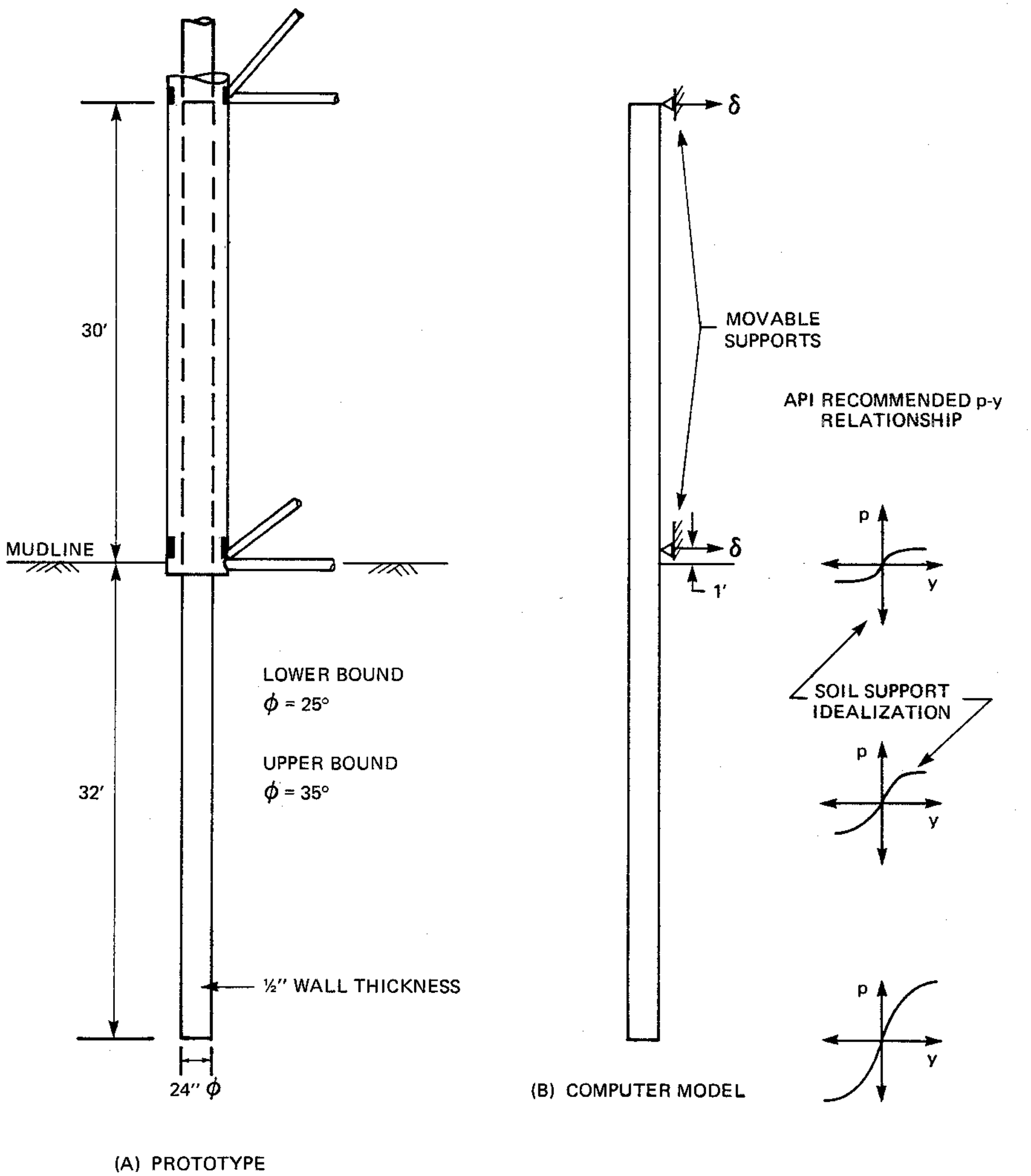
2.3 Sensitivity to Friction Angle

As part of the lateral pile design process, a set of soil parameters must be selected to define the site condition. These parameters are generally obtained from in situ or laboratory tests or local experience based on the design and performance of an existing structure in similar soils. In actual design, the parameters are often selected with a bias toward conservatism. However, for test design, the most probable values are needed and the effects of any variations need evaluation. It is for this reason that a brief sensitivity study was performed to investigate the effect of soil resistance prescription.

A free-head pile was arbitrarily selected to illustrate the consequence of pile head behavior under three assumed soil conditions. The three values of friction angle used were 30, 35 and 40 degrees. This range was selected because the interpreted friction angle for the sandy soil was close to 35 degrees. This sensitivity analysis was performed after the site investigation was completed. The clay layer was therefore included. The shear strength of the clay was 1.5 psi (1.0 kPa). Because the clay layer is secondary in effect to the sand, the shear strength was not varied for the three solutions.

The results of this sensitivity analysis are shown in Figs. 2-4 and 2-5 in the form of plots of applied load and maximum bending moment versus deflection plots at the lower strut level. Five degrees is probably the upper bound of uncertainty. The figures indicate about a ten-percent variation of reaction or bending moment for a 5-degree change in angle of internal friction. For the same change, the shear strength of the soil itself would change about 20 percent. This is a demonstration that the response of a free-head pile is somewhat insensitive to soil variation. With structural restraints added to the system the sensitivity is further reduced.

Two independent pile solutions were also performed to study the effect of a 3-foot (0.9 m) thick silty clay layer on a previously assumed uniform sand profile similar to the Seal Beach test case. The results in terms of bending moment and shear and pile-head behavior were not significantly different for the layered system when compared to the uniform case. A moderate variation was observed for the soil reactions at the location of the silty clay layer. The resistance of the clay layer at any particular deflection was much less than the sand resistance at the same depth. Therefore, the Seal Beach tests were primarily a sand-dominated experiment.



Ertec
The Earth Technology Corporation

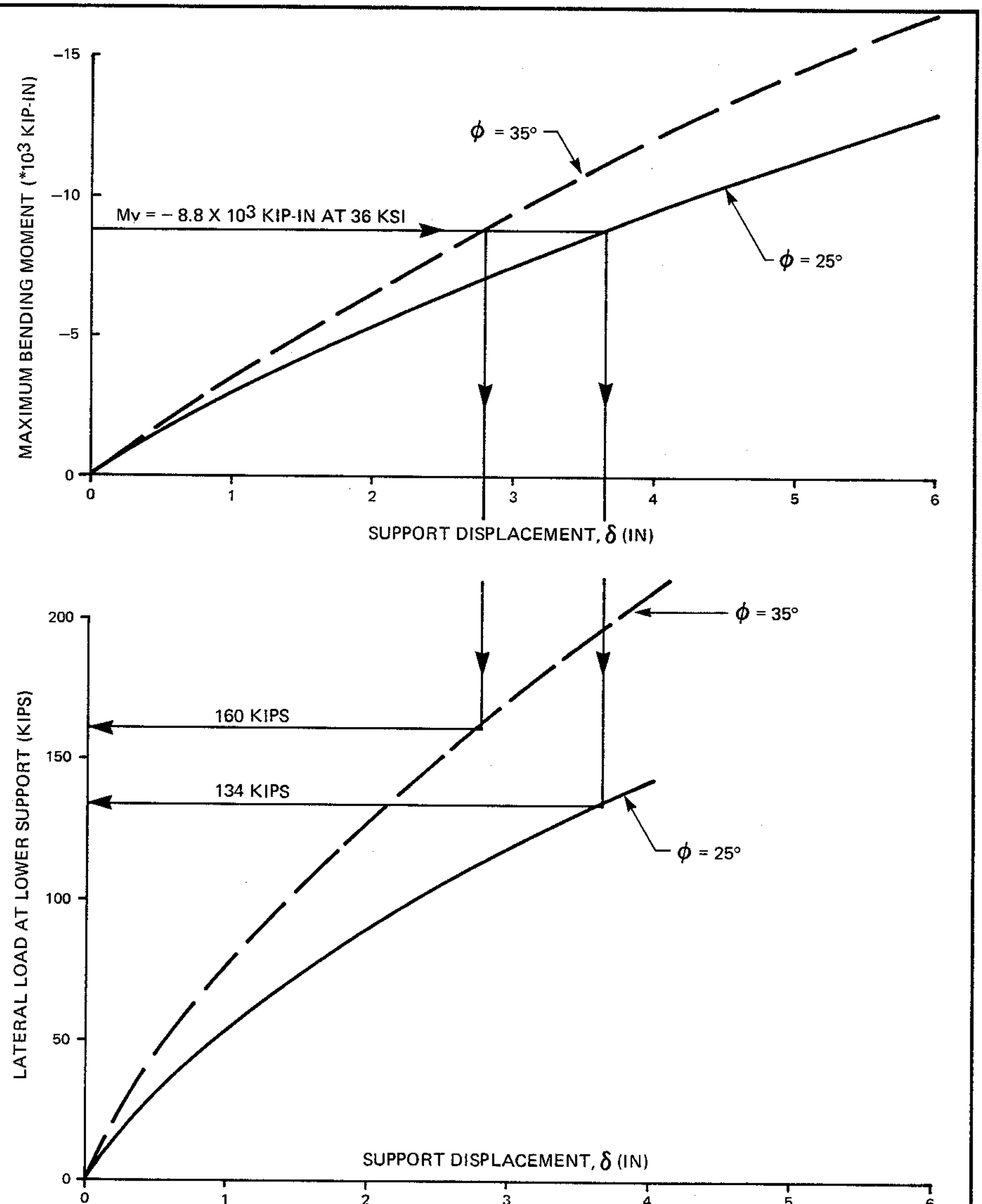
PROJECT NO.: 82-205

SEAL BEACH

PROTOTYPE PILE MAKE-UP AND MODEL USED IN BMCOL ANALYSIS

8-83

FIGURE 2-1



Ertec
The Earth Technology Corporation

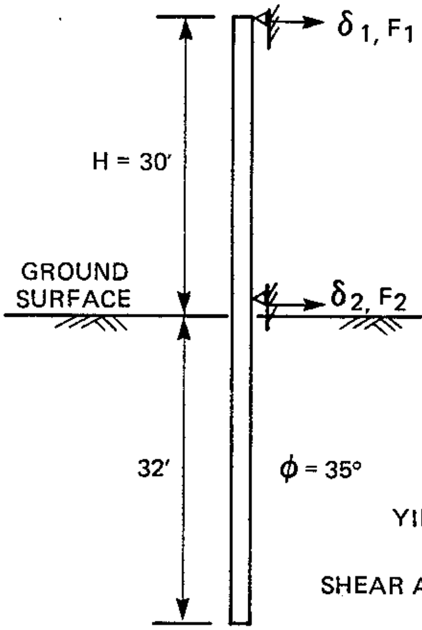
PROJECT NO.: 82-205

SEAL BEACH

SUMMARY OF MAXIMUM BENDING MOMENT AND LOWER SUPPORT REACTION FORCE FOR PROTOTYPE PILE

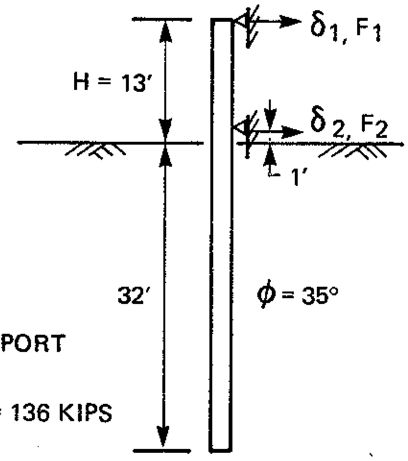
8-83

FIGURE 2-2



COMMON

PILE DIAMETER = 24"
WALL THICKNESS = ½"
YIELD MOMENT* AT LOWER SUPPORT
$(F_1 * H) = 8{,}800$ KIP-INCH
SHEAR AT LOWER SUPPORT $(F_1 + F_2) = 136$ KIPS



(A) PROTOTYPE CASE	ITEMS	(B) TEST CASE
−24 KIPS	F_1	−61 KIPS
160 KIPS	F_2	197 KIPS
2.8 INCHES	δ_1	3.5 INCHES
2.8 INCHES	δ_2	2.6 INCHES
1.0	$\frac{\delta_1}{\delta_2}$	1.44

*COMPUTED FOR STEEL STRESS = 36 KSI

Ertec
The Earth Technology Corporation

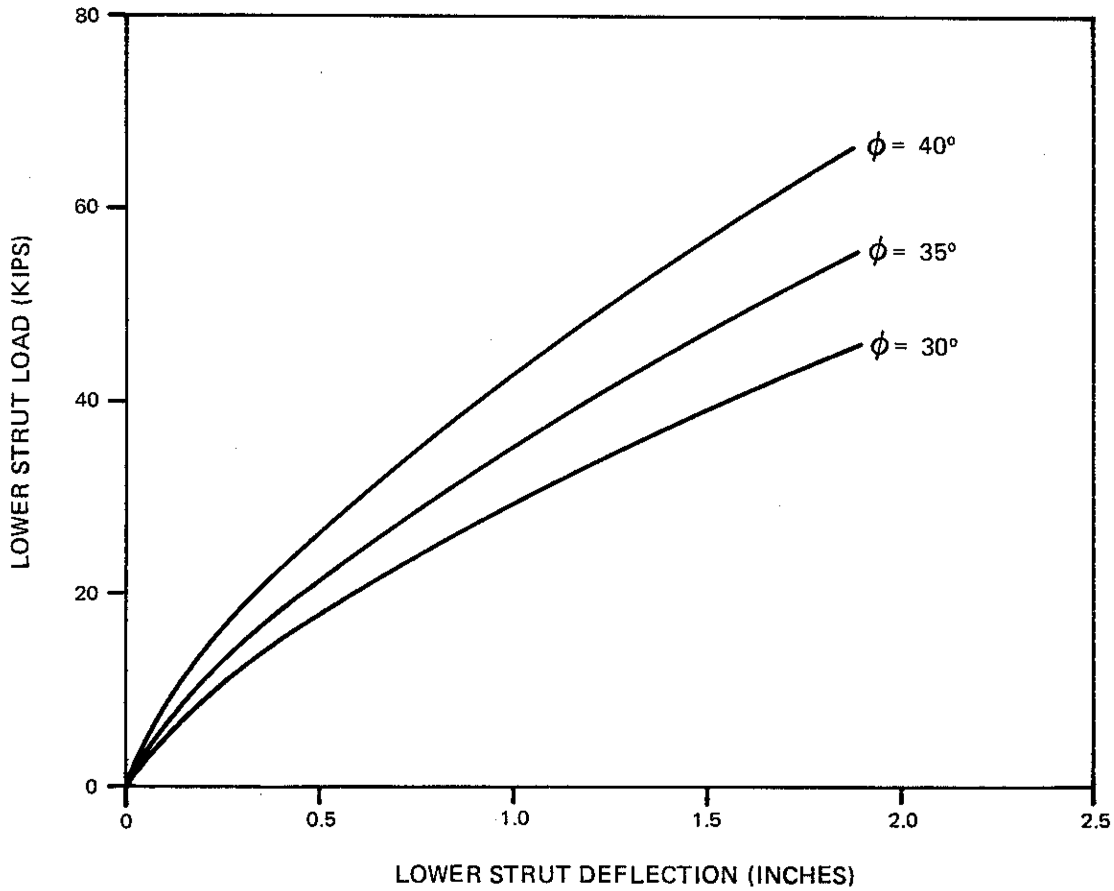
PROJECT NO.: 82-205

SEAL BEACH

COMPARISON OF DEFLECTION AND REACTION FORCE AT LOWER SUPPORT FOR PROTOTYPE AND TEST CASES

8-83

FIGURE 2-3



Ertec
The Earth Technology Corporation

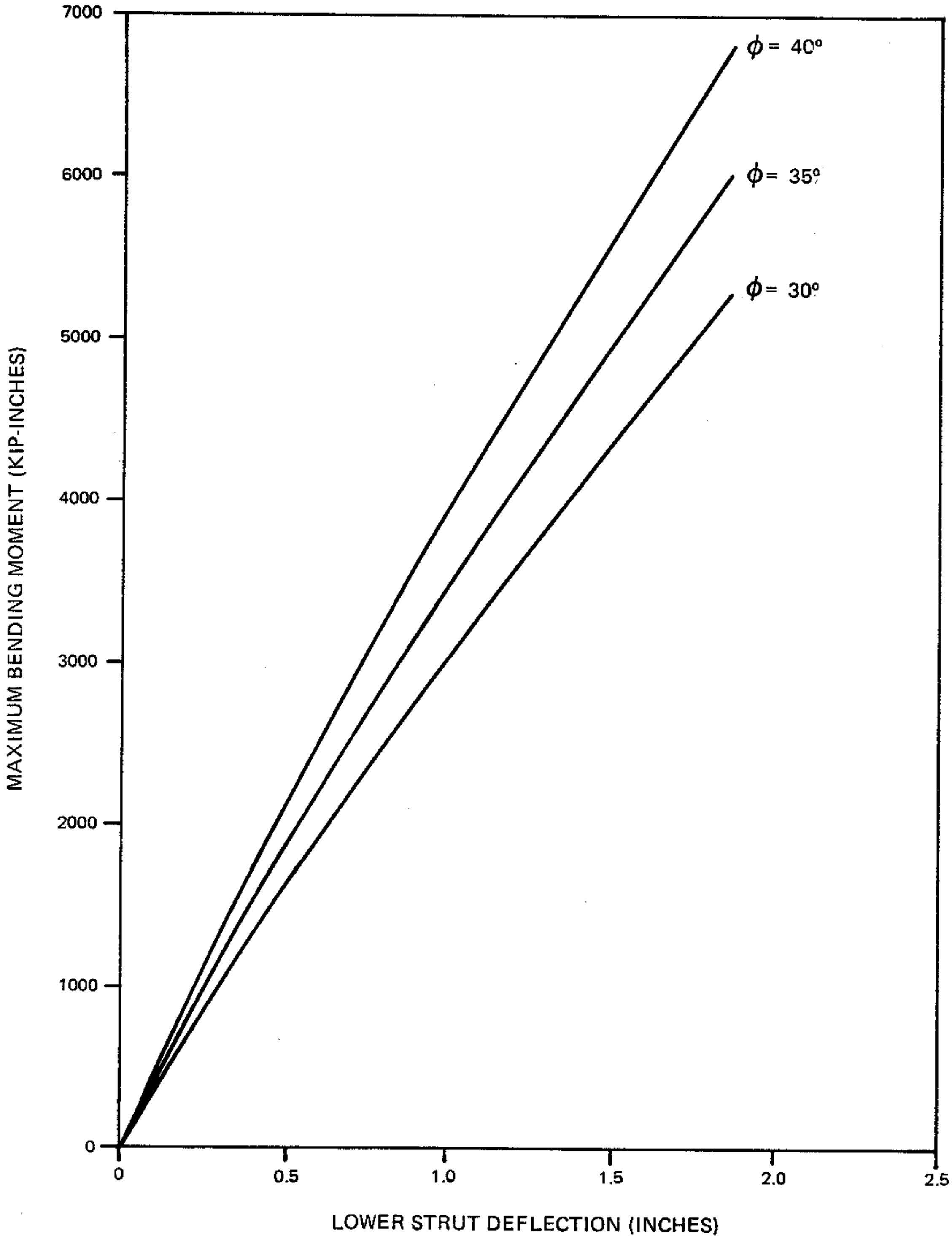
PROJECT NO.: 82-205

SEAL BEACH

COMPUTED LOWER STRUT LOAD-DEFLECTION RELATIONSHIPS FOR THREE DIFFERENT MAGNITUDES OF FRICTION ANGLE

8-83

FIGURE 2-4



Ertec
The Earth Technology Corporation

PROJECT NO.: 82-205

SEAL BEACH

COMPUTED MAXIMUM BENDING MOMENT FOR THREE DIFFERENT MAGNITUDES OF FRICTION ANGLE

8-83

FIGURE 2-5

3.0 GEOTECHNICAL INVESTIGATIONS

3.1 Background

The site of the lateral test program was originally selected for the previous NSF pile vibration tests (Ertec, 1981; Scott et al, 1982). One of the major objectives of the original study was to examine the effects of possible liquefaction on the soil-pile system during vibration. Therefore, a site was desired which exhibited a uniform, cohesionless soil deposit with the water table very close to the ground surface.

A site which conformed to the needs of the NSF program was found at the southern city limits of Seal Beach, Orange County, California. The U.S. Navy owns the property, which lies on the principal tidal inlet into Huntington Harbor at the eastern end of Anaheim Bay (Fig. 3-1). Permission from the Navy to use the property was granted in January, 1981, and extended through September, 1987.

The test site is on a small sandy, flat-bottom beach alongside the highway crossing. The width of the beach is approximately 150 ft (46 m), measured along the water line, and it is flooded regularly by tides. Elevation of high tide varies from 0.0 to 5.5 ft (0 to 1.7 m) with respect to the elevation of the ground surface at the location of the piles. A contour of tide variations, established during the NSF study, is shown in Fig. 3-2.

The soil conditions were evaluated for the NSF tests by performing two types of in-situ tests: (1) cone penetrometer tests with the Ertec electric friction cone penetrometer, and (2) continuous standard penetration tests with the standard split-spoon sampler. For completeness in this report, a boring log and cone soundings obtained during the NSF study are given in the Appendix, along with locations of boreholes and soundings with respect to the test piles.

The NSF site investigation program indicated that the soil at the Seal Beach test site consisted of 18 to 20 ft (5.5 to 6.1 m) of medium dense uniform silty sand overlying strata of silt, clayey silt, sand and siltstone. The test piles were subsequently located approximately 30 feet (9.1 m) away from the closest cone sounding. The soundings and soil boring did not encounter the clay layer found subsequently at this distance.

3.2 Supplemental Site Investigation Program

Additional geotechnical investigations were performed as part of the present study to supplement the existing NSF data. The additional work consisted of four cone penetrometer soundings and two boreholes in which high quality, continuous piston sampling was done. The objectives of the investigation were the following:

(1) To provide physical and mechanical soil properties for input into the API procedure for predicting and comparing lateral pile behavior.

(2) To thoroughly classify and document the site soils for subsequent application of any resulting design methods or modifications.

(3) To determine the densification effects of pile vibration from the previous NSF tests by probing very close to both the vibrated pile and the untested pile.

3.3 In Situ Testing

In addition to the three cone penetrometer soundings performed in the NSF study, four more soundings were done. Equipment used for the soundings consisted of the Earth Technology Corporation's electric friction cone penetrometer deployed with a newly-developed, trailer-mounted hydraulic frame (Fig. 3-3) for cone penetrometer testing. This unit enabled cone testing very close (1 foot; 0.3 m) to the test piles. The tests were completed on 24 August 1982.

The results and locations of the four cone soundings are shown in Fig. 3-4. As indicated in Fig. 3-4, the tests were performed along the centerline at locations 1 and 5 feet (0.3 and 1.5 m) from each of the two test piles. The computer-reduced plots in Fig. 3-4 present profiles of the soils in terms of unit cone resistance and friction ratio with depth. The unit cone resistance is an indication of a soil's strength and density as determined by a bearing-type failure. The friction ratio (ratio of unit side friction to unit cone resistance, expressed as a percentage) is an indication of soil composition. The low strength zones coinciding with the higher friction ratios are clear indications of a soft clay layer in all four soundings.

The clay layer is located at a depth of 5 to 8 ft (1.5 to 2.4 m) below the ground surface. The soil layers above and below the clay layer are uniform loose to medium dense silty sands which extend to an average depth of 20 ft (6.1 m). The soundings were terminated at the same depth (20 ft; 6.1 m). From the samples recovered in the soil boring, the medium dense silty sand continued to a depth of 32 ft (9.8 m). Scattered sea shells and interbedded thin silty clay and clayey silt layers were also detected at the deeper penetrations (beyond 20 ft or 6.1 m).

The cone logs clearly show the effect of pile vibration on the surrounding soils. Pile 1 was previously vibrated in the NSF tests. A higher cone resistance was measured near the ground surface at a distance of 1 foot (0.3 m) from Pile 1 as compared to the resistance measured at the same distance from the undisturbed Pile 2. The soil densification around Pile 1 probably was a direct result of the earlier vibration test.

3.4 Soil Sampling

The purpose of the soil sampling program was to recover high quality samples for laboratory testing. To accomplish the objective, two boreholes located approximately 12 feet (3.1 m) on each side perpendicular to the centerline of the two test piles, were drilled to depths of 32 ft (9.8 m) and 17 ft (5.2 m) for Boring 1 and 2, respectively. The exact borehole locations with respect to the piles are shown in Fig. 3-4.

The drilling and sampling operation was subcontracted to Pitcher Drilling Company of Daly City, California. A truck-mounted conventional rotary drilling rig, together with a 3-inch Osterberg piston sampler, was used. For Boring 1, sampling was continuous to a depth of 20 feet (6.1 m) and at 5-foot (1.5 m) intervals thereafter to termination depth. For Boring 2, sampling was continuous from the ground surface to termination depth.

The soil at this site can be divided into three generalized strata. The first stratum consists of a medium dense silty sand layer with scattered shell fragments and gravels which extends to about 5 feet (1.5 m) below the ground surface. A silty clay exists between the depths of 5 and 7 feet (1.5 and 2.1 m). The thickness of this silty clay layer is less than that indicated by the cone soundings. This must be the result of local soil variation since the soil borings were performed further away from the test piles as compared to the locations of the cone soundings. The next stratum, which occurs at a depth of 7 feet (2.1 m) to the boring termination depth of 32 feet, (9.8 m) again consists of medium dense silty sand with occasional interbedded silty clay and clayey silt layers. Scattered shell fragments are also found in the silty sand strata.

Special care was given to prevent excessive disturbance to the soil samples after recovery. The retrieved samples were all kept inside the Shelby tubes, sealed with wax, and transported vertically in a foam-padded container to the Earth Technology Corporation soil testing laboratory in Long Beach.

3.5 <u>Summary of Laboratory Test Results</u>

Soil identification and classification tests including grain size analyses, unit weights, and water contents, were conducted together with triaxial tests for strength determination. Atterberg limit tests were also performed for the cohesive soils present at the test site. The water content and submerged unit weight profiles obtained from the two borings are presented in Fig. 3-5. The results of particle size analysis for selected samples are shown in Figs. 3-6 and 3-7.

Two conventional triaxial compression tests were performed in the laboratory testing program. The number of triaxial tests were limited due to an earlier decision to derive soil strength parameters from more reliable in situ measurements such as the cone penetrometer tests. The two triaxial tests performed were (1) an isotropically consolidated - drained (ICD) test on a sand sample recovered from a depth of about 11 feet (3.4 m), and (2) an unconsolidated - undrained (UU) test on a clay sample taken from a depth of about 6 feet (1.8 m) below the ground surface. The results of the ICD test are shown in Fig. 3-8.

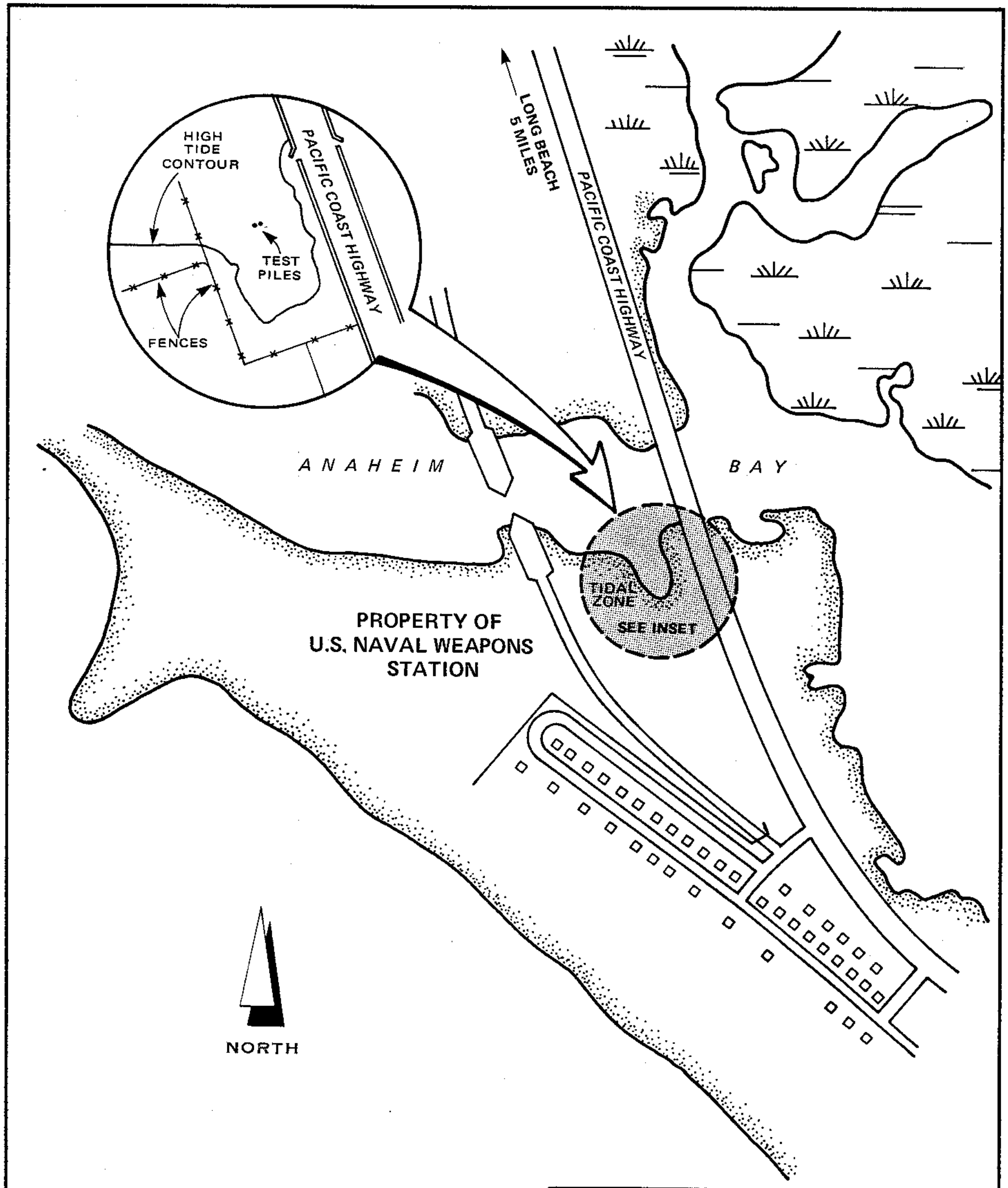
The results of the ICD test shown in Fig. 3-8 consisted of a stress-strain plot, volume change characteristics during soil shearing and a Mohr diagram indicating the shear failure condition. The ICD test was performed in three stages at three different cell pressures of 1.5, 3.0 and 6.0 psi (10.3, 20.7 and 41.4 kPa). The volume change plot shows some compaction in the beginning of the test. The angle of internal friction obtained from the test is about 42 degrees. This magnitude is thought to be high, possibly for several reasons; for example, confining pressure influence as discussed by Boutwell (1968). What is believed to be a more representative friction angle was subsequently derived based on cone data and is discussed later in this chapter.

The confining pressure used in the UU test was 1.0 psi (6.9 kPa). The shear strength, as interpreted from the magnitudes of confining pressure and failure stress, is about 1.5 psi (10.3 kPa). From the cone soundings, a shear strength value close to 1.5 psi (10.3 kPa) was also obtained by using an average measured cone tip resistance of 1.0 tsf (95.8 kPa) and an assumed bearing capacity factor (N_c) of 10. The bearing capacity factor was considered reasonable for this type of soil. This shear strength value was used in subsequent analysis to define the shear resistance of the silty clay layer.

3.6 Soil Parameters Used in Analysis of Pile Test Data

The soil parameters profile, selected for use in the analysis and interpretation of the pile load test data, is shown in Fig. 3-9. A friction angle of 35 degrees was used for the surficial and at-depth silty sand layers. Derivation of the friction angle was based on cone penetrometer records and empirical correlations as reported by Mitchell and Lunne (1978). The silty clay shear strength, as determined by the UU triaxial test, was 1.5 psi (10.3 kPa).

In order to apply the design chart published by Mitchell and Lunne, a constant submerged unit weight of 50 pcf (1.8 kN/m^3) was used. This value was derived from a careful interpretation of laboratory test results and measured cone resistance for the cohesionless soil. For the cohesive soil, a value of 54 pcf (1.9 kN/m^3) was obtained in the laboratory. For simplicity, a value of 50 pcf (1.8 kN/m^3) was also used for the 3 feet (0.9 m) thick silty clay layer. The strain at 50 percent of maximum stress was assumed to be 2 percent which is consistent with the recommendation given by Matlock (1970) for soft clay.



Ertec
The Earth Technology Corporation

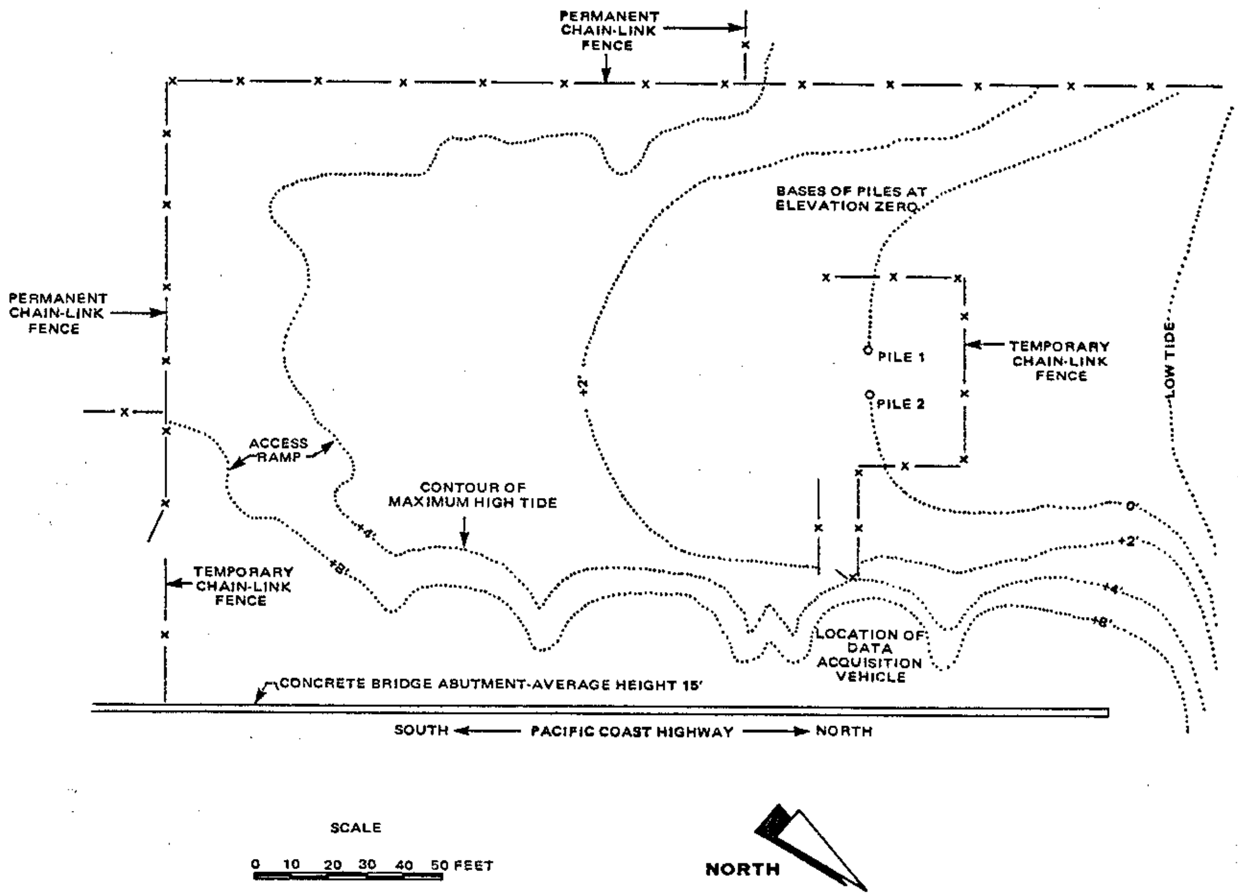
PROJECT NO.: 82-205

SEAL BEACH

SEAL BEACH SITE LAYOUT

8-83

FIGURE 3-1



Ertec
The Earth Technology Corporation

PROJECT NO.: 82-205

SEAL BEACH

PLAN VIEW OF SITE, SHOWING CONTOURS OF TIDAL VARIATIONS

8-83

FIGURE 3-2

Compiled by ______ Drawn by ______ Checked by ______ Approved by ______

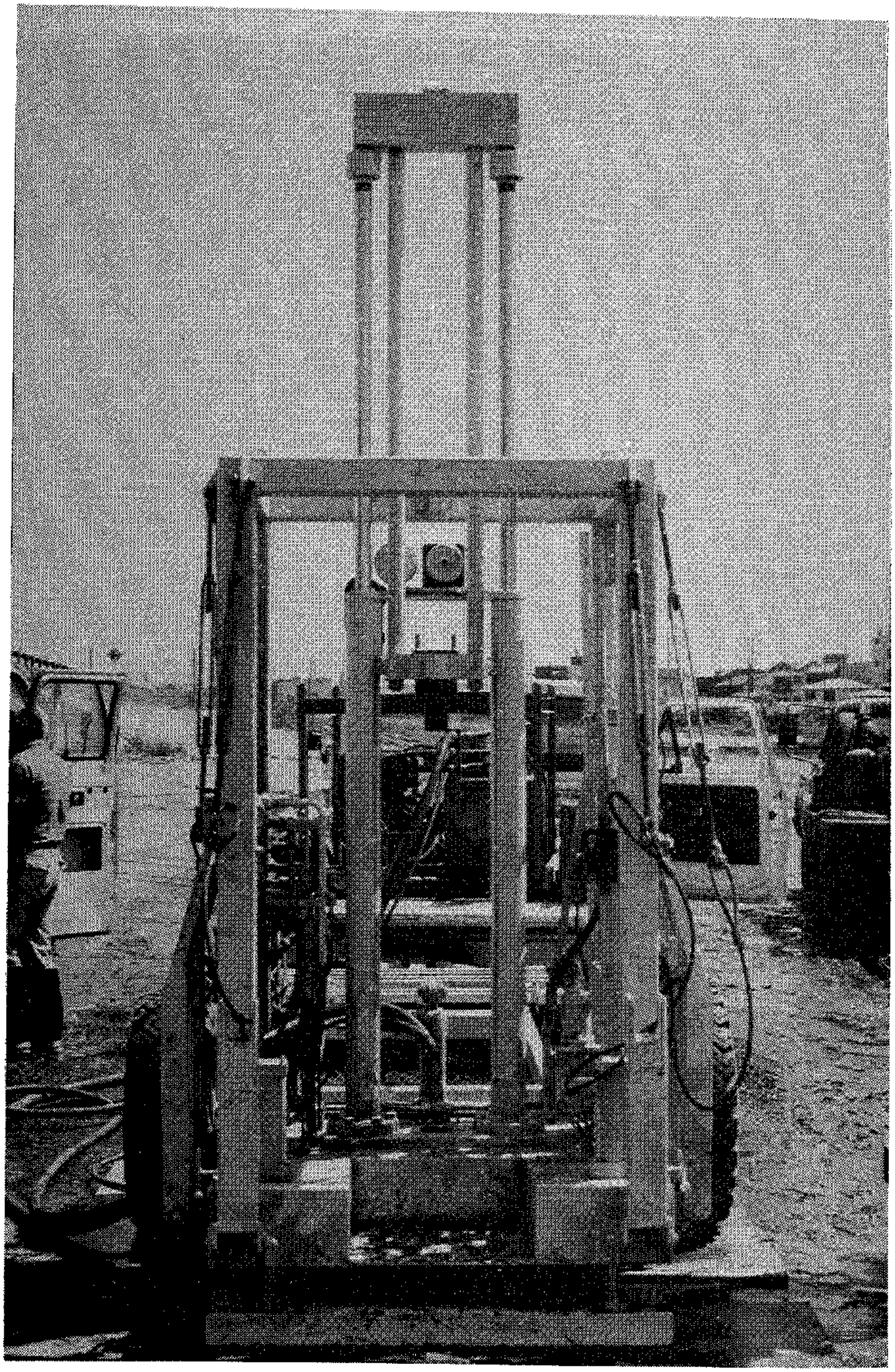
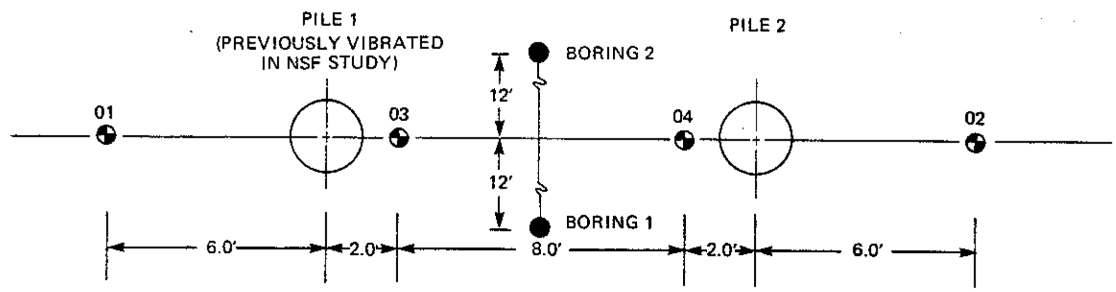
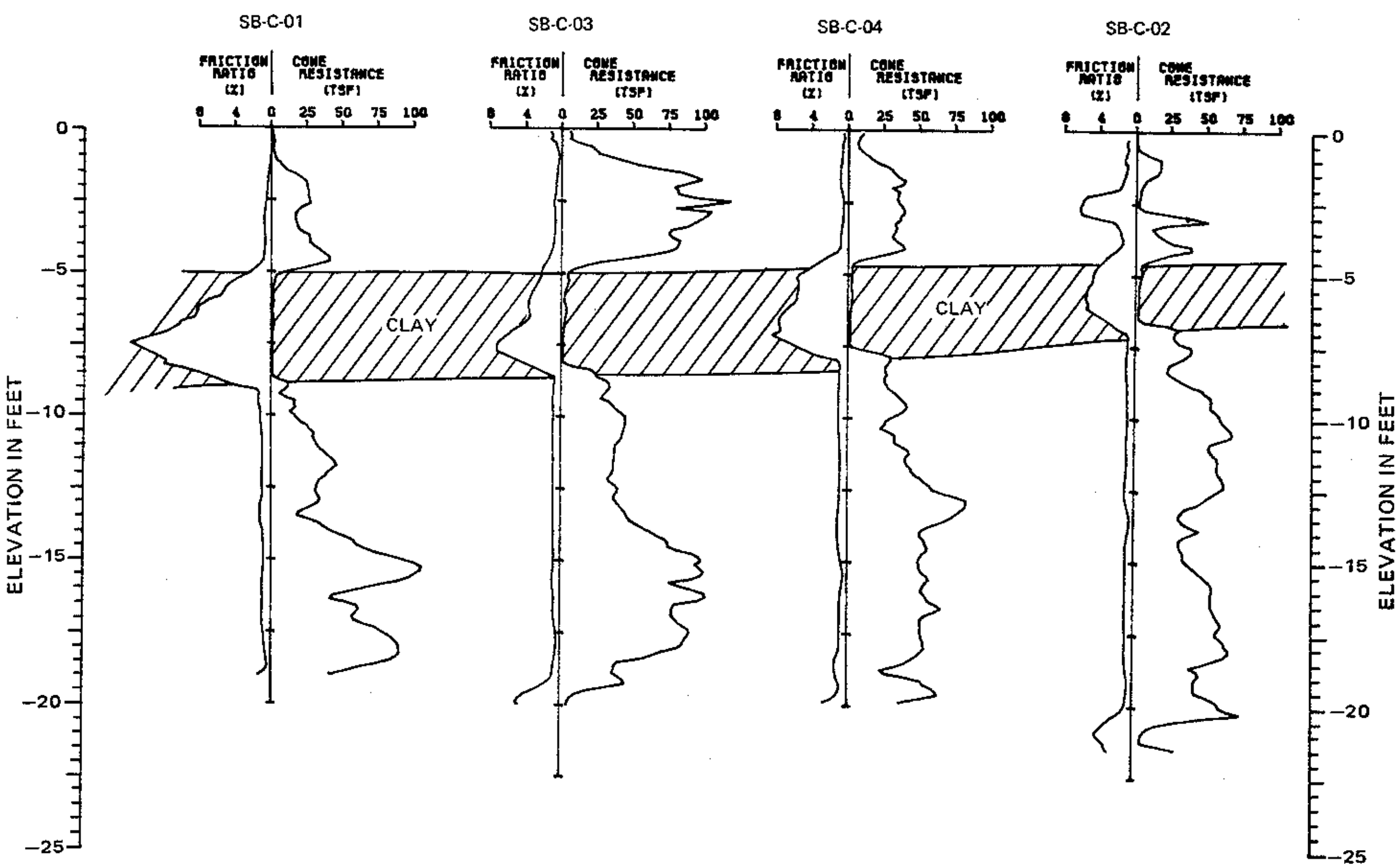


FIGURE 3-3 PORTABLE CONE PENETROMETER TESTING (CPT) UNIT

PLAN VIEW OF PILES, CPT SOUNDING AND SOIL BORING LOCATIONS



PROFILE OF CPT SOUNDINGS



Ertec
The Earth Technology Corporation

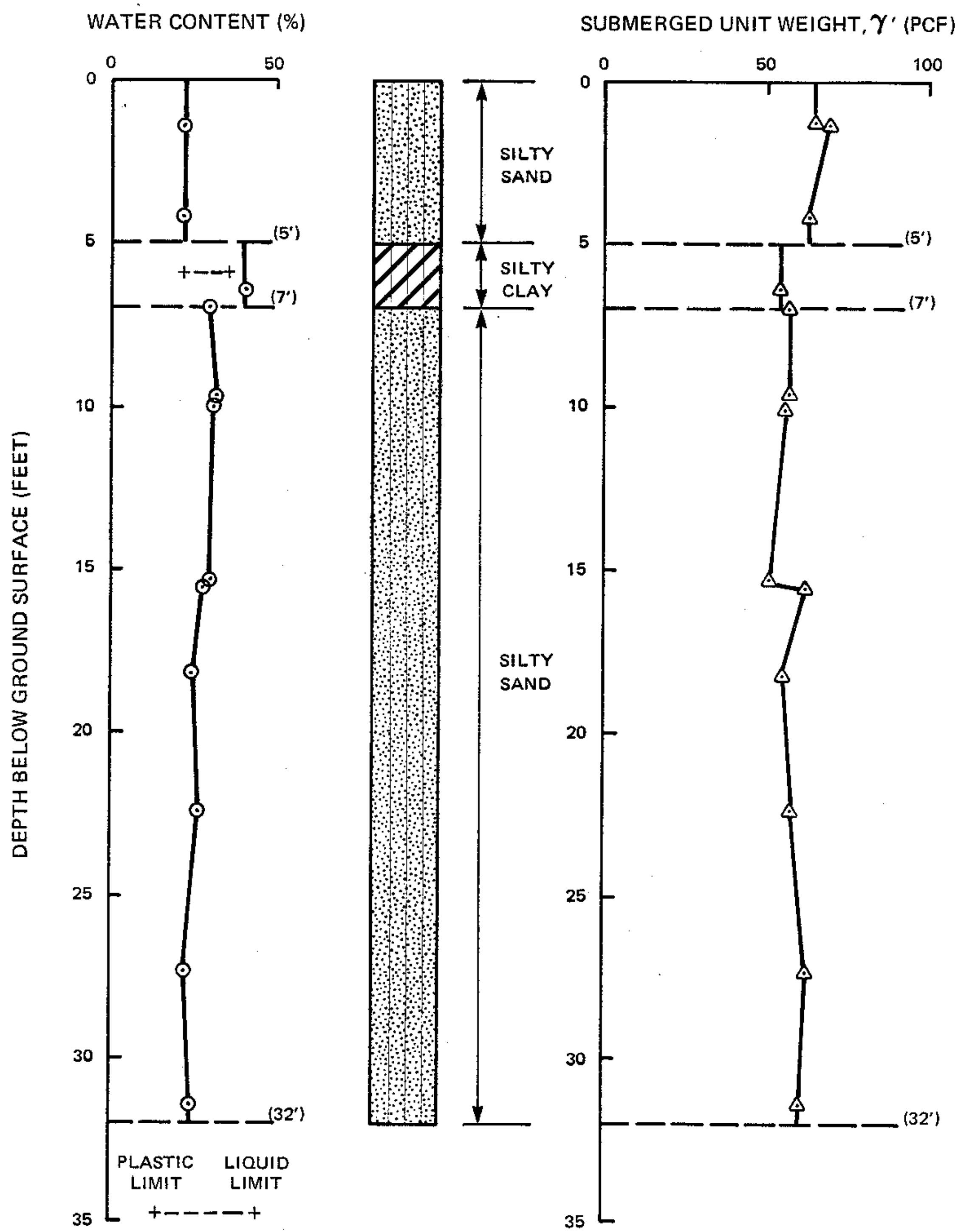
PROJECT NO.: 82-205

SEAL BEACH

SOIL PROFILES AS IDENTIFIED BY
CPT SOUNDINGS PRIOR TO
LOAD TESTS

8-83

FIGURE 3-4



Ertec

The Earth Technology Corporation

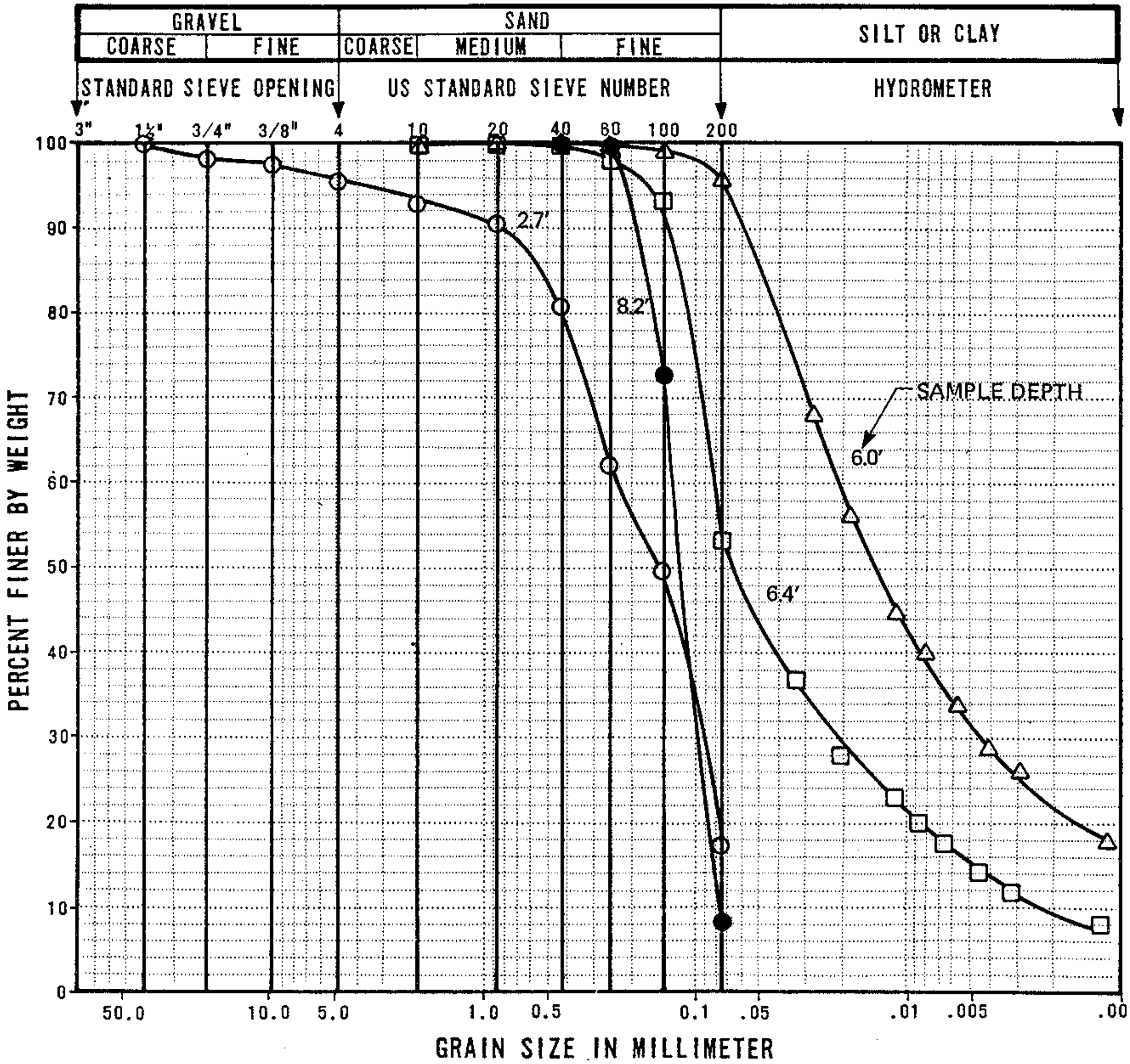
PROJECT NO.: 82-205

SEAL BEACH

LABORATORY DETERMINED WATER CONTENT AND SUBMERGED UNIT WEIGHT PROFILES

8-83

FIGURE 3-6



Ertec
The Earth Technology Corporation

PROJECT NO.: 82-205

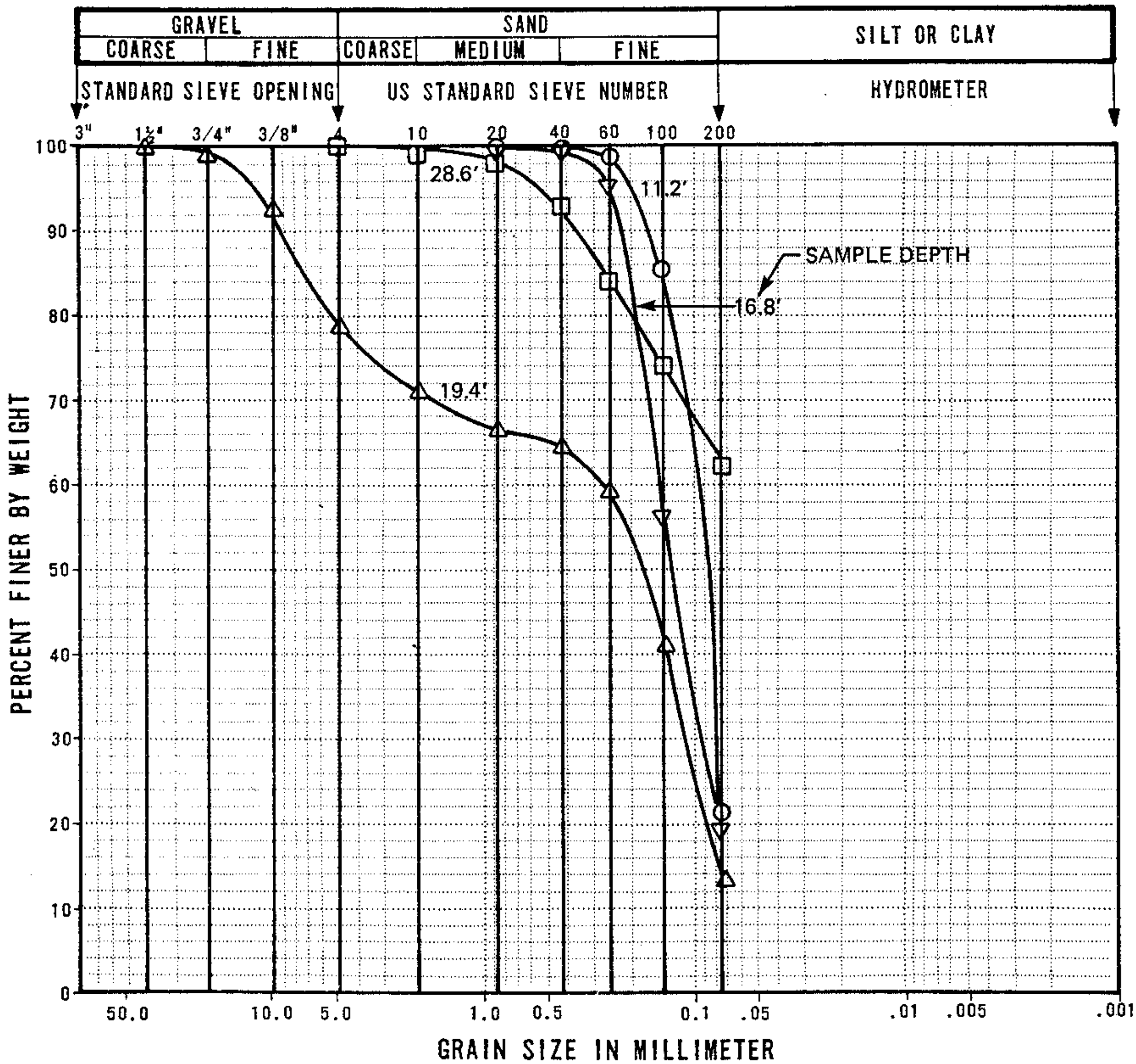
SEAL BEACH

RESULTS OF PARTICLE SIZE ANALYSIS FOR SAMPLE DEPTHS BETWEEN 0 AND 10 FEET

8-83

FIGURE 3-6

Compiled by ______ / Drawn by ______ / Checked by ______ / Approved by ______



Compiled by ______ / Drawn by ______ / Checked by ______ / Approved by ______

Ertec — The Earth Technology Corporation

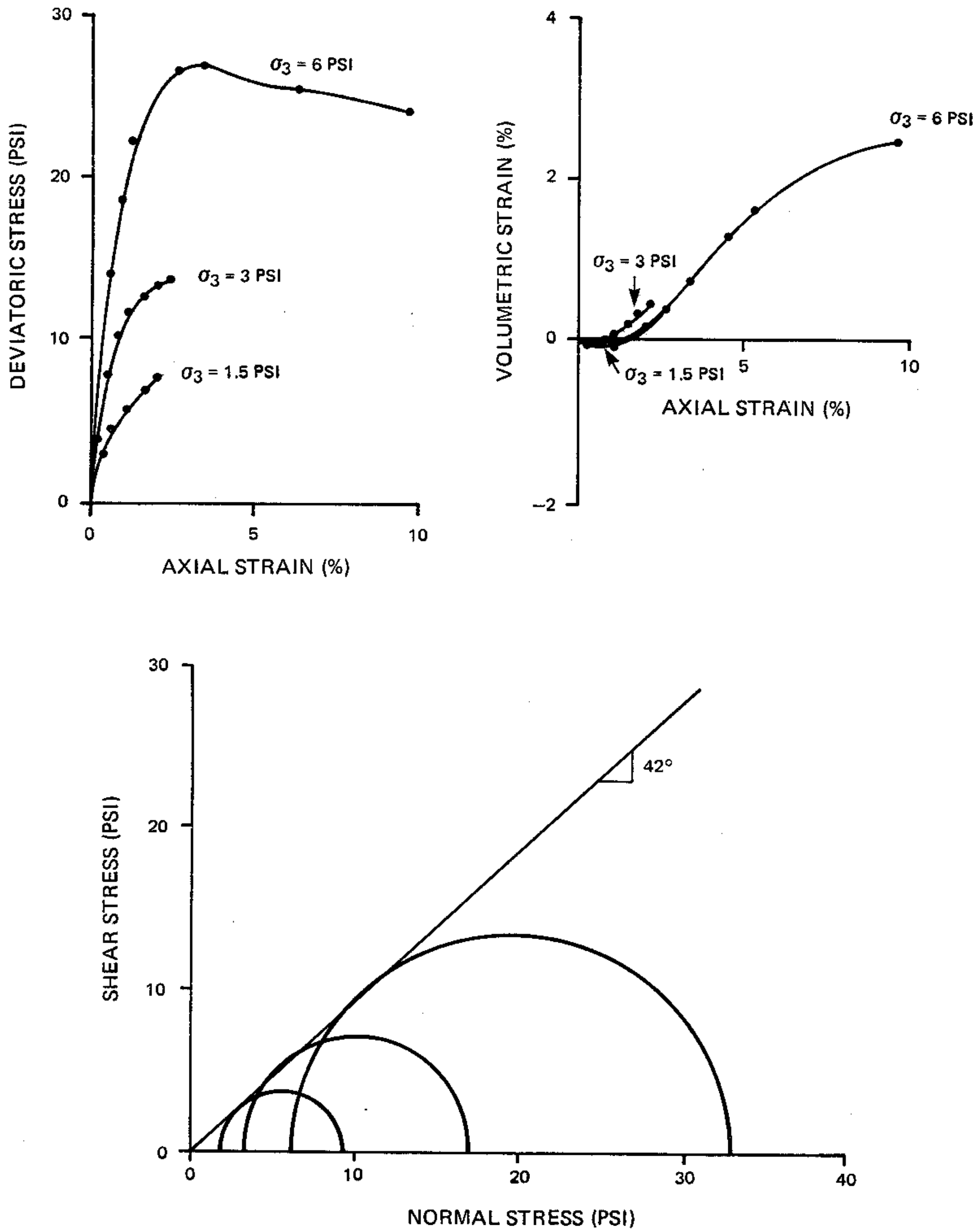
PROJECT NO.: 82-205

SEAL BEACH

RESULTS OF PARTICLE SIZE ANALYSIS FOR SAMPLE DEPTHS BETWEEN 10 AND 30 FEET

8-83

FIGURE 3-7



Ertec
The Earth Technology Corporation

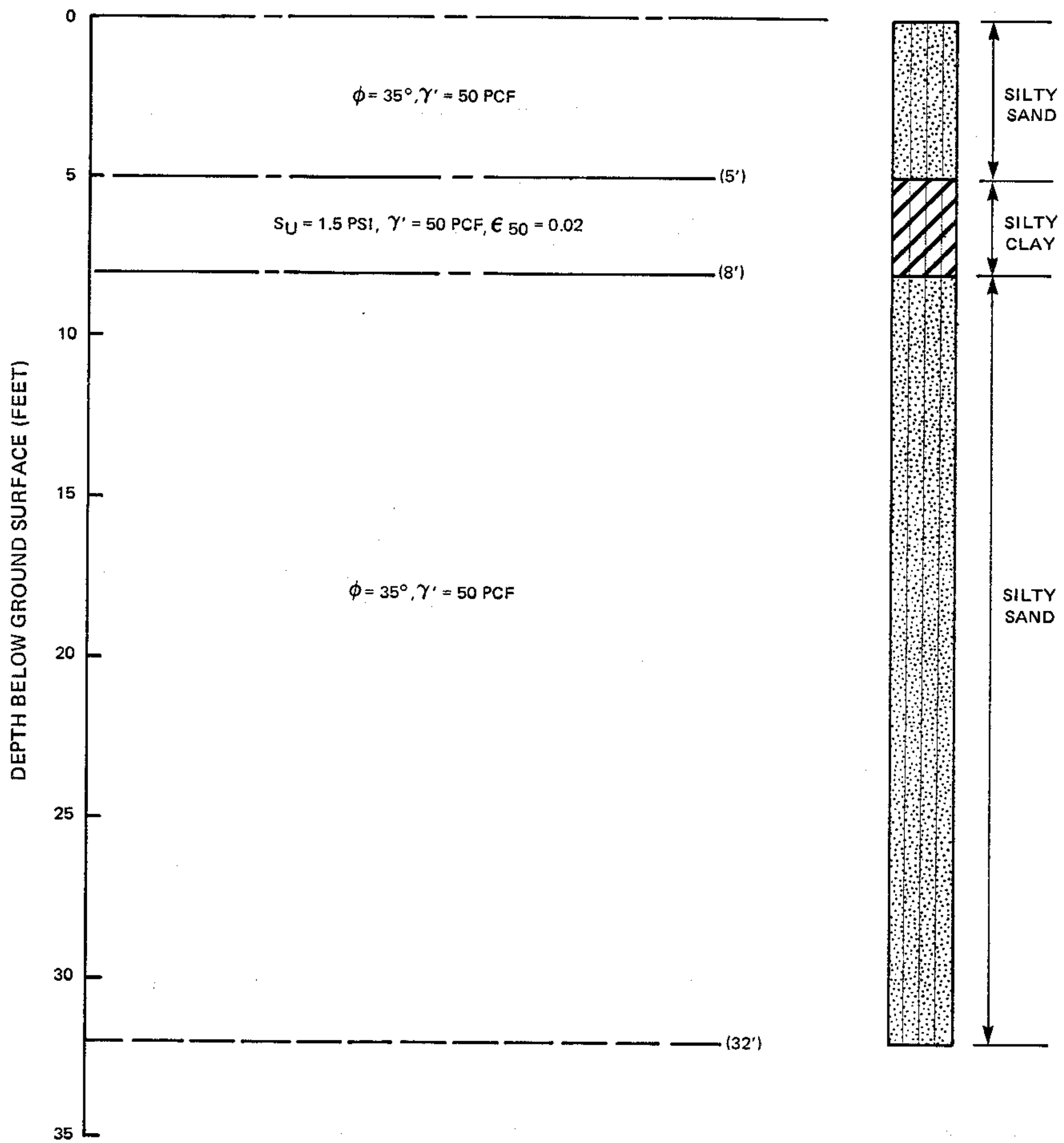
PROJECT NO.: 82-205

SEAL BEACH

MULTI-STAGE ISOTROPICALLY
CONSOLIDATED-DRAINED
TRIAXIAL TEST RESULTS
FOR SAND

8-83

FIGURE 3-8



Ertec
The Earth Technology Corporation

PROJECT NO.: 82-205

SEAL BEACH

SOIL PARAMETERS PROFILE USED IN ANALYSIS OF PILE LOAD TEST DATA

8-83

FIGURE 3-9

4.0 PHYSICAL TEST SYSTEM

4.1 General

A diagram of the Seal Beach load test set-up is presented in Fig. 4-1. The two test piles are sections of grade B steel pipe, 24 inches (61 cm) in outside diameter, having a wall thickness of 0.5 inch (1.3 cm) and approximately 45 feet (13.7 m) long. The piles were driven as 40-foot (12.2 m) sections to an embedment of 32 feet (9.8 m) for the NSF study, and were extended 5 feet (1.5 m) for use in the present program. One pile, designated as Pile 1, was tested in vibration during the NSF program. Pile 2 was left in its original, as-driven condition.

The major components of the physical test system were as follows:

(1) Loading struts, top and bottom

(2) Hydraulic rams, power supply, pump, and controls

(3) Instrumentation

  a. Strut loads
  b. Lateral deflection
  c. Bending moments in piles
  d. Pore pressure in soil
  e. Pile Inclination

(4) Data acquisition system

4.2 Load Frames and Hydraulics

The loading system was designed to permit free-head (FH), fully restrained-head (FRH), and partially restrained-head (PRH) conditions near the ground surface that would approximately simulate the prototype cases. Loads were applied by hydraulic rams at 1.0 foot (0.3 m) above the ground surface. The elevation of the top restraint, when in use, was 13 feet (4.0 m) above the ground surface. All points of loading were blunt knife edges bearing on flat surfaces to closely simulate point load conditions. The loading system provided control of only the relative displacement between the two piles.

Top views of the load-strut assemblies are shown in Fig. 4-2. The bottom load strut was a telescoping assembly of plates and 5 and 6-inch (12.7 and 15.2 cm) heavy-wall standard pipes, straddling the two test piles. The three 1/2-inch (1.3 cm) plates welded across the telescoping unit were used to mount the hydraulic rams and act as knife edges acting against bearing strips. The stiffeners shown in Fig. 4-2(c) were fabricated from 1/2-inch (1.3 cm) plates and a 4 x 12-inch (10.2 x 30.5 cm) rectangular tubing with a wall thickness of 1/2-inch (1.3 cm). The stiffeners were welded to the two piles to serve as bearing surfaces during loading and were also used to distribute the applied load to the piles. Shims were installed loosely during actual testing between the knife edges and bearings to minimize clearance for faster load reversal.

The top strut across the pile tops was a single 6-inch (15.2 cm) standard heavy-walled pipe with one end attached to a 1.5-inch (3.8 cm) diameter high-strength ACME threaded rod. A 30-inch (76.2 cm) diameter wheel and nut was threaded over the rod together with a bearing plate and thrust bearing on each side of Pile 1. The bearing plate was translated by rotating the wheel in order to adjust the pile top deflection to simulate a particular head-restraint. Once the desired deflection was achieved, matching ACME locknuts were used to secure the wheel and thus prohibited additional lateral movement of the pile top. The components of the top and bottom load struts are also shown in the diagram in Fig. 4-1 together with location of instrumentation during test set-up.

Forces required for loading the bottom strut were provided by the four 50-kip (222 kN) hydraulic cylinders mounted between plates of the telescoping framework. The cylinders were activated by the same two-stage, 12 h.p. diesel-powered pump as is used with the Earth Technology Corporation portable cone penetrometer rig mentioned in Section 3.3. Hydraulic controls were assembled such that directions of loading were governed by a solenoid directional control valve, controlled manually or by computer command from the data acquisition system. The rate of deflection was controlled by manual hydraulic flow control valves. A schematic of the hydraulic loading system is shown in Fig. 4-3.

4.3 Instrumentation

Strut Loads. Loads in the top and bottom struts were monitored with the use of strain-gaged load cells welded to the struts so as to form a continuous section of each pipe strut. The load cells were designed, instrumented, and calibrated in-house by personnel from the Earth Technology Corporation.

Strain gages were applied to the insides of the cells and, after wiring, the cells were sealed against intrusion of moisture. Gages were attached and wired in a full Wheatstone bridge arrangement.

<u>Deflection</u>. Deflections were measured electronically by the use of four DC-LVDTs, two mounted at each pile at elevations of 39 and 87 inches (99 and 246 cm) from the ground surface. The LVDTs were supported by four wooden reference beams. The core of each LVDT was attached on one side by a small cable, and on the other side to a constant-tension cable stretched by a weight over a pulley (refer to Fig. 4-1). The LVDTs for Pile 2 were backed up by extended range dial indicators at both elevations of measurement. To enable calculation of deflections at the top and bottom points of loading, the LVDT wires and dial indicators were attached to a vertical reference bar rather than to the pile wall directly. The reference bars were attached to the piles by pins at the elevations of loading; thus, deflections at those points could be calculated from the two simultaneous measurements on the pinned bar.

<u>Moment</u>. A new approach for instrumenting the test piles for measurement of pile wall strain was developed for use in the previous NSF study. The method consisted of potting prefabricated strain-gaged steel tubes with epoxy into sealed sections of square steel tubing, previously cleaned and welded into the inside of the pile wall. The method and the strain

gage bridge configuration for each depth of measurement are described and illustrated in Fig. 4-4.

The epoxy was manufactured by Emerson & Cuming. The selected resin "Stycast 2057" and the catalyst "24LV" produced a low viscosity epoxy with high strength and roughness. The compressive strength of the epoxy was reported to be 13,000 psi (90,000 kPa) by the manufacturer. The set-up time was also quite slow and was important in the potting process because large quantities of epoxy were mixed and poured.

In order to obtain a near-zero initial reading on the strain gage bridge, the resistance of the individual strain gages were adjusted by polishing the surface with mild abrasive. This resistance adjustment was made with the strain gages attached to the steel tubes but before the water proofing coating was applied. The final resistance of individual strain gages and the bridge resistance were also periodically checked before and after installation in the test piles.

Pile 1 was instrumented in the above manner for the NSF tests, and the other pile was left uninstrumented until the degree of success of the first was ascertained. The system performed satisfactorily during the NSF tests, even at low magnitude loads. Pile 2 was instrumented on 30 August 1982 in the identical manner, and functioned well thereafter.

<u>Pore Pressure</u>. Four piezometer tubes were installed at locations outside of each pile, as shown in Fig. 4-1. Each

tube was tipped with a porous cone-like element over which a stainless steel well screen was fitted. For the piezometer tubes adjacent to the test piles, another fitting at the above-ground end of the piezometer tube led to electrical pore pressure transducers, and included a shut-off valve for assistance in saturating the system. These piezometers were placed to measure pore pressure changes at a depth of 9.8 feet (3 m) below the ground surface.

<u>Pile Inclination</u>. Pile-head inclination was measured periodically by a 20-second level bubble and a micrometer screw mounted on an aluminum channel. The channel was supported by the micrometer screw and two other ball feet 24 inches (61 cm) away. Spacers were secured at fixed locations on a platform welded to the pile top to insure precise and repeated placement of the supports of the channel. The level was balanced by the adjustment of the micrometer. Each measurement consisted of two readings, taken with the micrometer first facing east and then west. The inclination of the pile top was then calculated from these two measurements and the distance L between the support points.

$$\text{Pile Inclination} = \frac{E-W}{2L}$$

where

E and W = East and West micrometer readings.

This arrangement is used to cancel instrument adjustment errors and thereby gives a highly reliable check on pile slope that is independent of all other methods of measurement.

4.4 Data Acquisition

The combination of transducers to measure strut loads, deflections, moments in both piles, and pore pressures in two of the piezometer rods totaled 25 channels of data that could be sampled by an automatic system. The equipment selected to obtain and record the data consisted of the following:

(1) Microcomputer: Hewlett-Packard HP85F

(2) Scanner and Voltmeter: Hewlett-Packard 3497A

(3) Plotter: Hewlett-Packard 9872A

(4) Plotter: Hewlett-Packard 7046A

(5) Precision strain gage balance box: Ertec

With the above system, the tests and all data acquisition could be computer-controlled, with optional manual control of loading and data acquisition by operator command. All transducer cables were interfaced with the data acquisition hardware via a panel box manufactured specifically for the Seal Beach tests. The panel box contained power supplies for all transducers, analog meters for visual observation of LVDT measurements, and a manual loading control. In addition, the panel provided switching from automatic to manual data acquisition modes so that absolute readings of strain transducers could be obtained independently by use of the precision bridge balance box. A simple schematic of the data acquistion system is given in Fig. 4-5.

Readings of the strain gage bridges were made before, during and after the experiments using the precision strain gage balance box. Since no zero balance circuitry was employed, these readings represented the absolute imbalance of the strain gage bridge. By comparing the bridge readings with the readings made before pile testing, and applying calibration constants, the absolute values of load and bending moment were determined. Since data acquisition was controlled, at almost all times, by the HP voltmeter and microcomputer, the manual balance box readings were used only for confirmation of the data collected by the computer-controlled digital system.

Data Sampling and Storage. All sampled field measurements were permanently stored on a cassette in the HP microcomputer. A dump print of the information stored in the cassette was made at the end of the field test. This dump print was filed for safety separately from the cassette. Occasional output from the built-in HP microcomputer printer were also available during field testing. These print-outs included information such as strut loads, pore-pressure, LVDT deflection and bending moment for all gage stations along the two test piles. This information was used for test control and preliminary evaluation of pile behavior.

During actual testing, manual control of loading and data acquisition was used only in the static test and the first and last cycles of each cyclic test. For pile loading in the manual mode, a complete data set from all instruments was collected by entering a command on the computer key board. The speed of a full scan of all channels plus storing the data varied from 10 to 20 seconds. The varying sampling rate was due to a flaw in the software and was a recognized limitation of the computer system. At one point during actual testing, this flaw caused a failure of the loading system to reverse the load after the pile had reached the deflection limit.

For cyclic testing, a partial data set including only loads, deflections and pore pressures but omitting bending moments was automatically recorded and stored at peak deflections before each reversal of loading. The time required for this process varied from 5 to 10 seconds. When a complete data set was desired, the loading operation could be switched to the manual mode and a full scan of all channels performed. As mentioned earlier, this option was used in the first and last cycles of each cyclic test.

Field Display. During each test, the data were also observed visually by printouts of full or partial data sets and by use of two plotters. One plotter was dedicated to continuously graph bottom strut load versus bottom deflection of Pile 2. The second plotter graphed digital moment data versus depth for both piles, so that moment curves could be observed as the test progressed.

Zero Readings. In order to establish a set of zero readings, a sweep of all the instruments was recorded and stored on the microcomputer using the digital HP scanner and voltmeter. This sweep of data recording was performed with the loading assembly in place just before the first load test. This set of zero readings in terms of raw voltages was used throughout the test program such that a fixed datum could be used to reference all subsequent changes in transducer and bridge circuit outputs.

4.5 Pile and Load Cell Calibrations

Eight strain gage stations were used to measure bending moments along a portion of each of the test piles. Each station consisted of four active gages with pairs subjected to equal and opposite strains. In this study, calibration of these strain gage stations were done by the following equation:

$$\frac{E_o}{E} = F\varepsilon \quad \text{. . . . .} \quad (4.1)$$

where

E_o = Output voltage

E = Excitation voltage

F = Gage factor (specified by manufacturer)

ε = Mechanical strain

The bending moment was then determined from the following well known relation:

$$\varepsilon = \frac{My}{EI} \quad \text{. . . . .} \quad (4.2)$$

where

M = Bending moment

y = Distance from neutral axis to the point of strain measurement

EI = Flexural stiffness of pile

The flexural stiffness (EI) of each test pile was calculated to be 85 x 10^6 kip-in (9.6 x 10^6 kN-m) which included the additional area of the strain gage tubes and protective channels. A Young's modulus of 29,000 ksi (200,000 kPa) was used.

This calibration method was expected to yield an accuracy of about ± 3 percent or better. This magnitude of precision was sufficient because there was no intention to obtain p-y relationships directly from the bending moment curves.

The load cells for the load struts had been calibrated in a hydraulic press. A 20-ton (178 kN) capacity load cell and the precision bridge balance box described earlier were used to take the readings. Several cycles of loading were applied during calibration to eliminate material hysteresis. Loading, in compression only, was done in 5-ton (44.5 kN) increments up to about 20 tons (178 kN). Unloading was done with the same steps. At each level of loading, the imbalance of the strain gage bridge was recorded by the precision bridge balance box. By comparing these readings with the zero reading taken with no load, the absolute values of load versus balance box units were obtained.

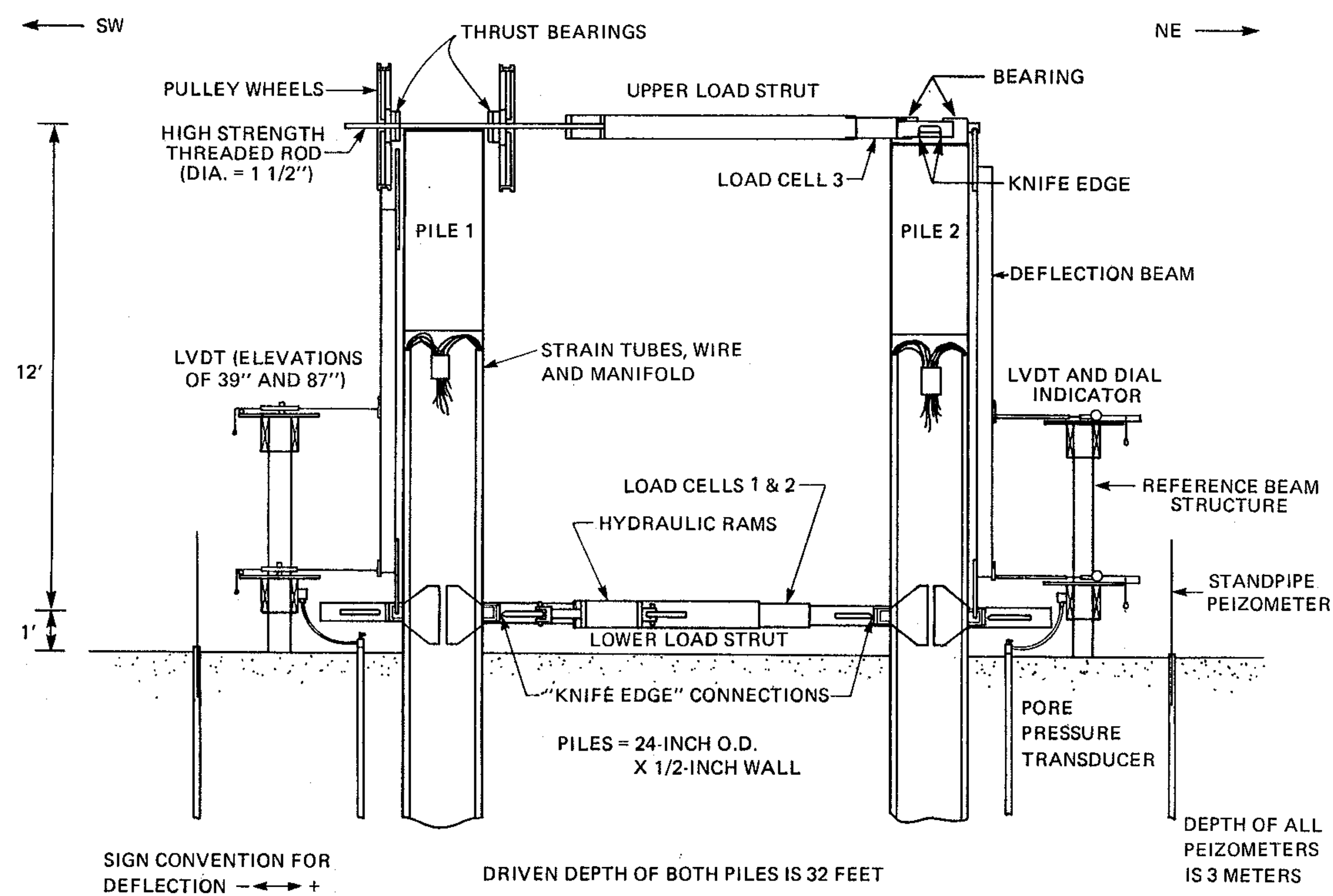
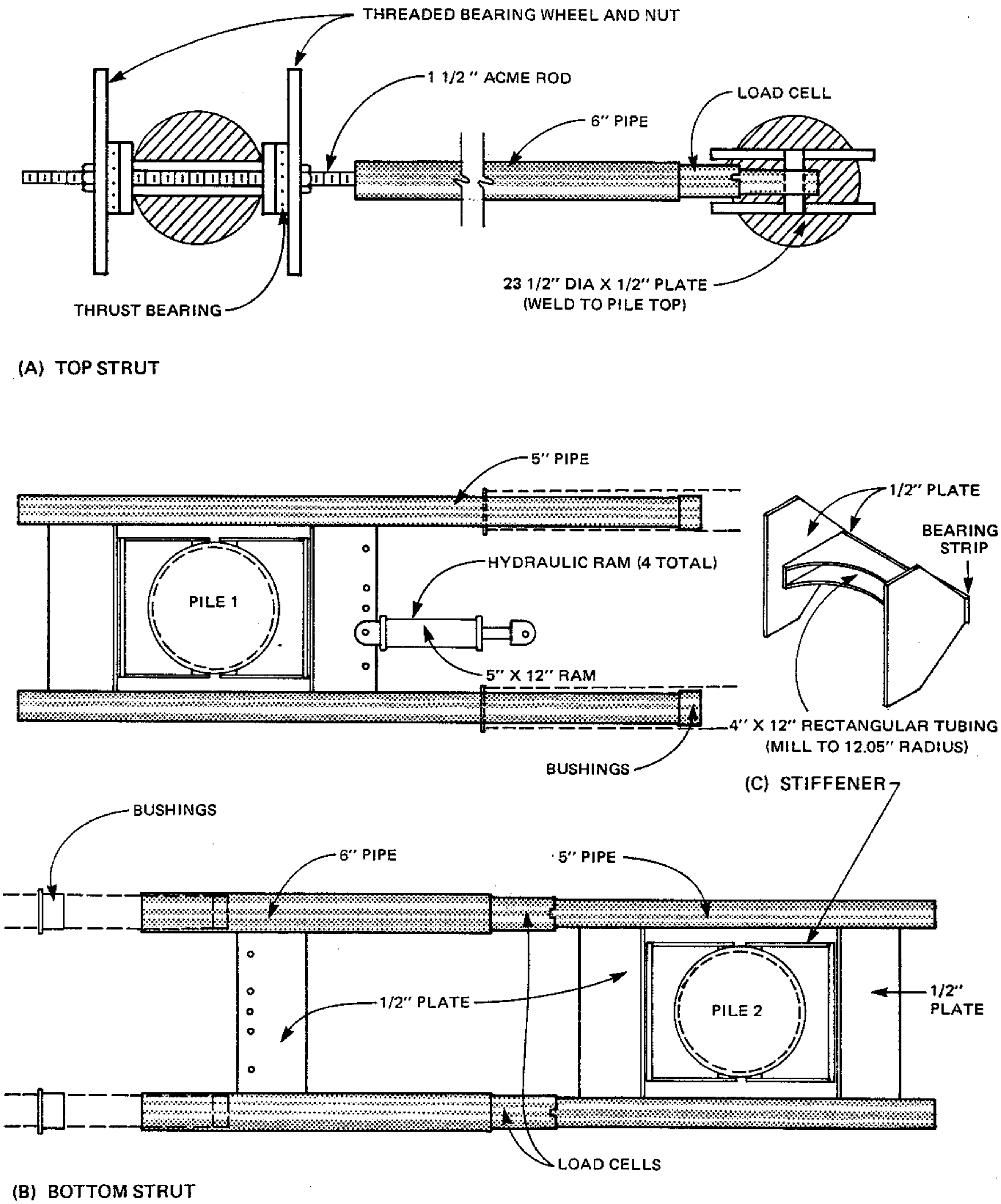


FIGURE 4-1 TEST SET-UP, LOOKING TOWARDS N.W. (SEE ALSO FIGURE 5-1)



Ertec
The Earth Technology Corporation

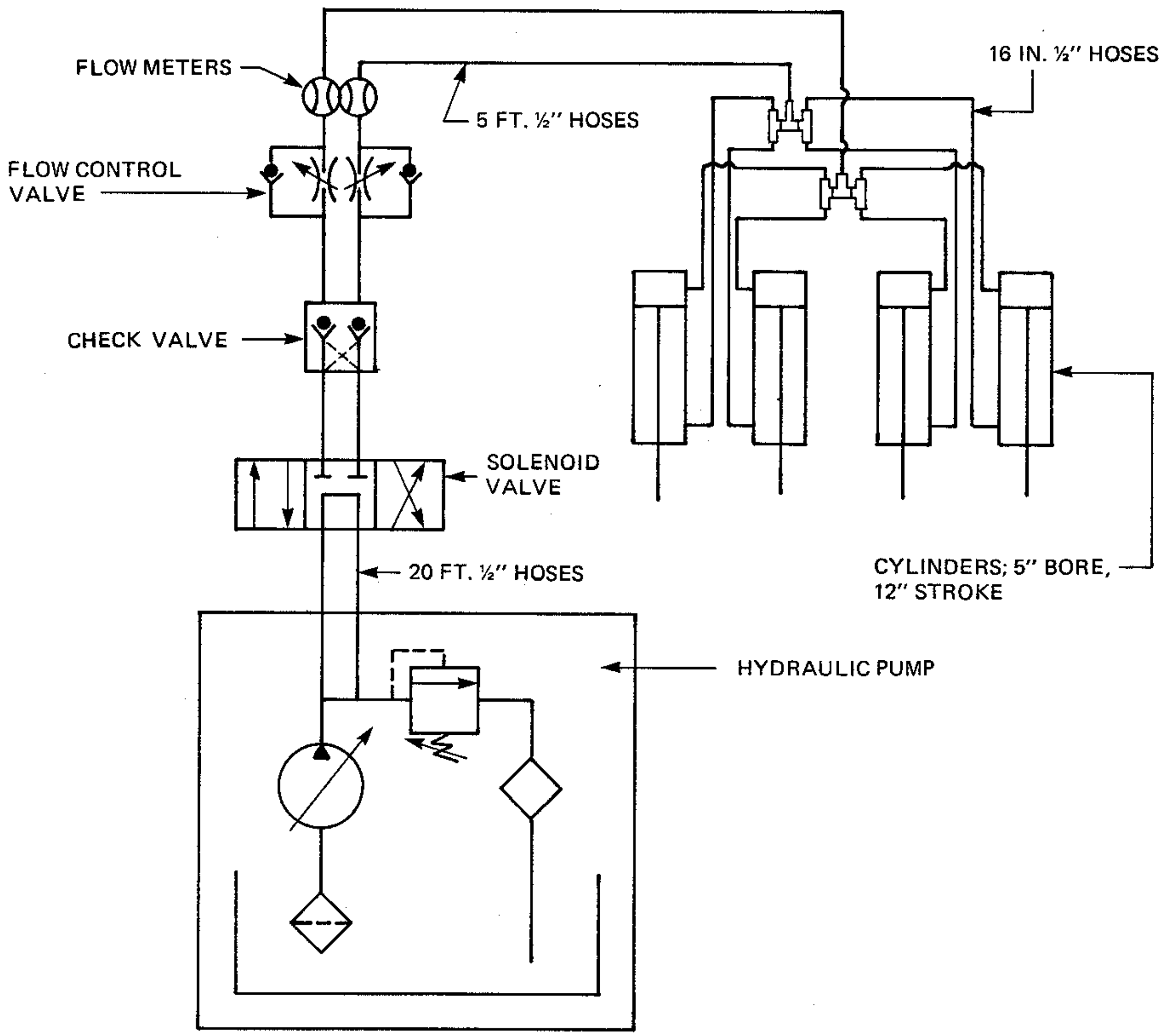
PROJECT NO.: 82-205

SEAL BEACH

TOP VIEW OF LOAD STRUT ASSEMBLY

8-83

FIGURE 4-2



Ertec
The Earth Technology Corporation

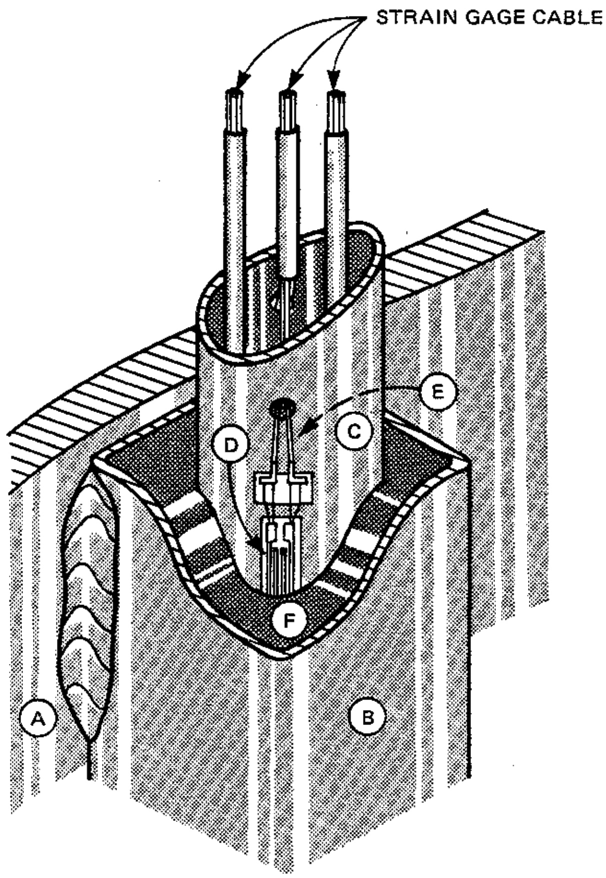
PROJECT NO.: 82-205

SEAL BEACH

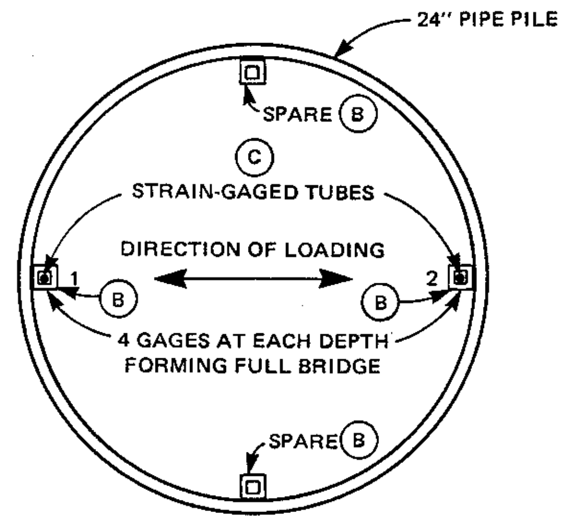
SCHEMATIC OF
HYDRAULIC LOADING SYSTEM

8-83

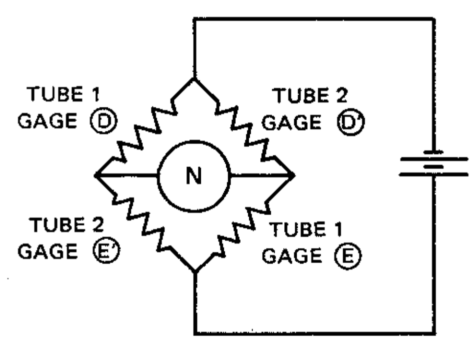
FIGURE 4-3



CUTAWAY VIEW OF ONE STRAIN GAGE TUBE



CROSS SECTION OF INSTRUMENTED PILE



CONFIGURATION OF STRAIN GAGE BRIDGE
(1 OF 8 IN PILE)

PROCEDURE FOR INSTRUMENTING PILE

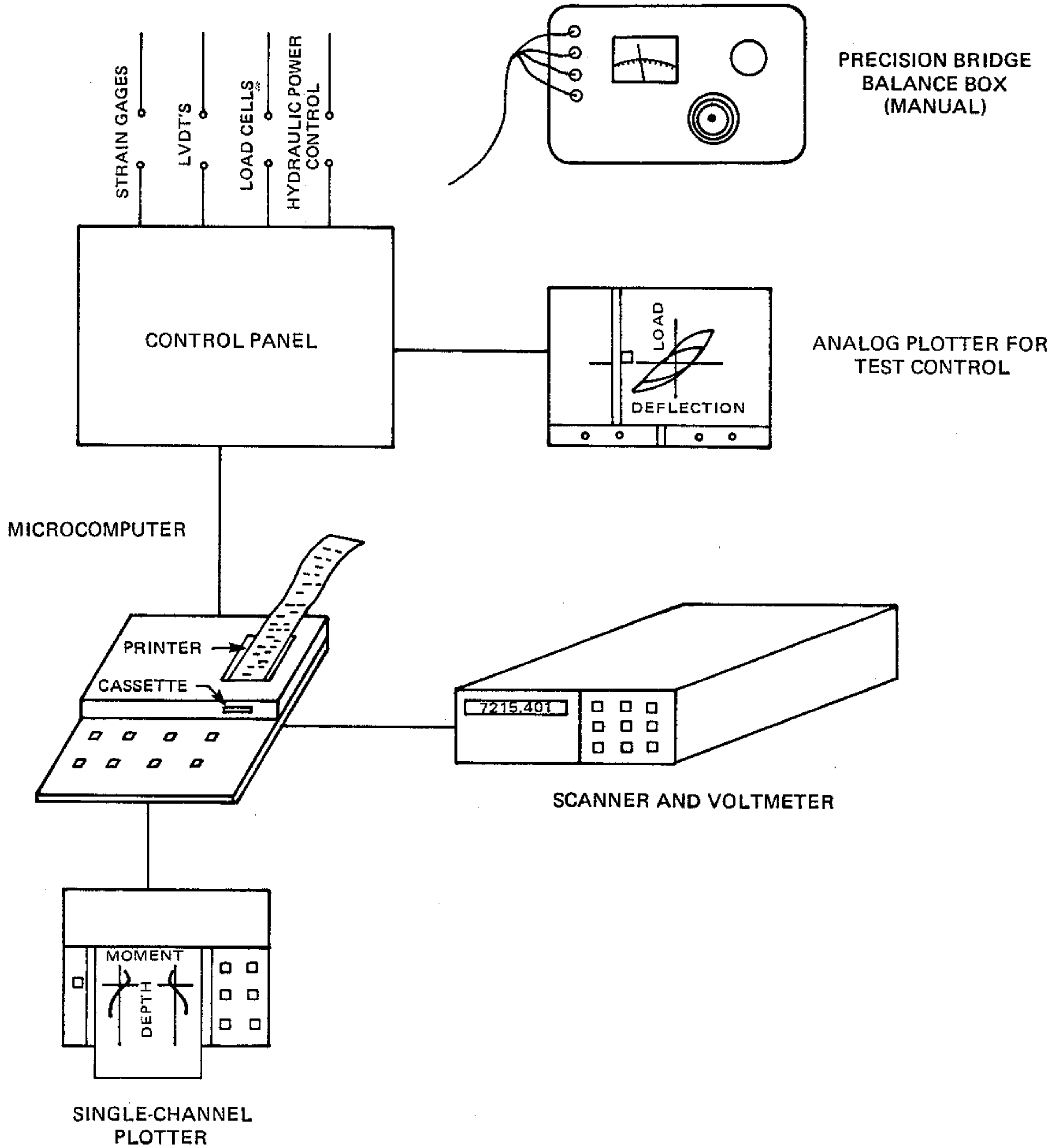
1. WELD 1 1/2" SQUARE STEEL TUBES (B) TO INSIDE OF 24" DIAMETER STEEL PILE (A)
2. PREFABRICATE 1" DIAMETER STEEL TUBE (C) BY EPOXYING 8 PAIRS STRAIN GAGES (D) AND (E)
3. PUT 2 STRAIN-GAGED TUBES INTO 2 OPPOSITE SQUARE TUBES WITH EPOXY COMPOUND (F)

Ertec The Earth Technology Corporation	PROJECT NO.: 82-205 SEAL BEACH

STRAIN GAGE CONFIGURATION

8-83

FIGURE 4-4



Ertec
The Earth Technology Corporation

PROJECT NO.: 82-205

SEAL BEACH

SCHEMATIC OF
DATA ACQUISITION SYSTEM

8-83

FIGURE 4-5

5.0 FIELD TESTS

5.1 General

The Seal Beach pile load tests were conducted during the period 22 to 24 September 1982. For all but the first day's program, the tests were coordinated with the tide schedule so that sea water was flooding the location during testing. However, when the water depth around the piles exceeded 1.5 ft (0.5 m), testing was not possible due to risk of moisture damage to the lower deflection-measuring instruments. Although the load cells on the bottom strut were sealed, the strut was hoisted above the water level during high tides to reduce the chance of moisture damage.

A photograph of the test layout is shown in Fig. 5-1. Two vehicles remained on site at all times during the three days of the program: a van which housed all data acquisition equipment, and a truck which carried the hydraulic pump and diesel engine power supply. Electrical power was supplied to the data acquisition equipment by a 3.2 KW gasoline-powered generator.

5.2 Test Control

All the Seal Beach pile tests were deflection-controlled tests, with applied loads left variable. Loads were applied by pushing the piles apart (major loading) and pulling them together (minor loading). For all cyclic loading, the ratio of major to minor deflections was 3:1 as a rough approximation of directional storm wave forces. The deflection of the lower LVDT on Pile 2 was used to control testing.

Three types of pile-head restraint were employed during the program: partial, full, and zero (free head) restraint. The partial head-restraint was intended to simulate, within the limitations of the physical test system, the prototype loading condition explained in Chapter 2, and was used for static loading only. Increments of load were applied at the lower load strut, and the threaded bearing wheel on the top strut was adjusted so that the deflection of the pile top was about 1.4 times that at the lower point of loading. The fully restrained case was utilized for cyclic tests, and consisted of locking the top strut so that no movement was allowed while loads were applied below. For the free-head tests, which were also cyclic tests, the top strut was freed completely.

5.3 Test Sequence

A summary of the load test sequence is given in Table 5-1 and a graphic visualization of the tests is shown in the summary plot of load versus deflection in Fig. 5-2. This plot was taken directly from the field record of lower strut load and the LVDT deflection at an elevation of +39 inches (99 cm). This field record, produced for Pile 2 only, was also displayed on an analog plotter during actual pile testing.

The program began on Wednesday, 22 September 1982 with a static test with a partially restrained-head condition. Loading was controlled in 0.1-inch (0.25 cm) increments of bottom deflection, and unloading to zero load was achieved in 0.2 inch (0.5 cm) increments.

Since the purpose of the first test was to check out the loading assembly and the instrumentation, the applied load was increased until the deflection at the lower strut of Pile 2 was about 0.7 inch (1.8 cm) which corresponded to a measured maximum bending moment of less than 500 kip-in (56 kN-m) or about 9 percent of allowable maximum stress of 24 ksi (165,000 kPa). This allowable bending stress corresponded to a calculated moment of 5860 kip-in (662 kN-m).

As shown by Test Series 1 in Fig. 5-2, a sawtooth pattern is observed for the load-deflection relationship at the lower strut for this first restrained-head test. A permanent offset of about 0.3 inch (0.8 cm) was also measured at the lower strut level at the end of this test. The sawtooth behavior was a result of the testing procedure. Three sequential steps were involved in applying each increment of loading:

(1) The top deflection was fixed by the locking nuts adjacent to the threaded wheels.

(2) Hydraulic rams were then activated to apply an incremental load until the deflection at the lower strut level reached a magnitude of about 0.1 inch (0.3 cm).

(3) The locking nuts were loosened and the wheel turned to allow a slow progressive lateral movement of the pile top until the desired ratio of top to bottom load strut deflection of about 1.4 was reached.

The loading path of each individual sawtooth was caused by the action of item (2) above. The unloading path was a result of the lateral pile top movement and resulting relaxation caused by the action described in item (3). Since the pile-head load-deflection relationship was not continuous, in terms of p-y characteristics, some load-unload effect was occurring at each step.

Free-head tests, Series 2 through 6 in Fig. 5-2, were initiated the next day. The first loading was done, in increments of 0.2 inch (0.5 cm) past the previous day's maximum movement to a major deflection of 1.0 inch (2.5 cm) from the original starting position. Loading was then reversed to zero deflection and continued to a minor deflection which corresponded to about one-third the major value. By this procedure the limiting deflections were set for the series of cyclic tests, which proceeded thereafter. A total of five cyclic free-head tests were performed.

Series three through six of the free-head test took place on Friday, 24 September 1982. The free-head cyclic tests were resumed at increasing levels of deflection until the maximum allowable pile stress of 24 ksi (165,000 kPa) was reached. The maximum deflection corresponding to this stress was 2.32 inches (5.9 cm), measured by Pile 2. For each level of deflection the loads were cycled until they stabilized.

At the conclusion of free-head cycling, the top strut was locked, and cyclic tests were started. A total of three cyclic tests, Series 7 through 9 in Fig. 5-2, were performed. In the first cyclic test, the piles were displaced 0.2 inch (0.5 cm) past the deflection at the end of the free-head test. The same increment of deflection was used for the second and third cyclic tests. The deflection limits were set in the same manner as the free-head tests.

During the third cycle of the first test, a flaw in the computer logic delayed the reversal of loading after the pile had reached the major deflection. Consequently, the piles were continually loaded to a stress of about 21 ksi (145,000 kPa) before the load was reversed manually. In the second cyclic test, the maximum stress was also about 21 ksi (145,000 kPa). Since the piles had not reached the allowable limit of 24 ksi (165,000 kPa), they were deflected another 0.2 inch (0.5 cm) on the third cyclic test. However, at the major deflection of this third test, a bending stress of 34 ksi (234,000 kPa) was already measured. At this stress level, the test piles were at a near-yield condition. Subsequent moment curves suggest that slight permanent yielding did occur.

The major deflection of Pile 2 at 34 ksi (234,000 kPa) was 0.9 inch (2.3 cm). During this test series, virtually no change in load was observed with increasing numbers of cycles, at all three levels of loading.

The final test conducted on the Seal Beach piles was a repetition of the first test, a partially restrained-head test. To more closely simulate continuous loading, increments of loading were kept very small. In this way, readjustment of the threaded bearing wheel could be minimized for each deflection increment applied by the lower strut, resulting in a more continuous load-deflection pattern exerted by the pile on the soil.

5.4 Loading Rate

The loading rate was controlled by the capacity of the diesel-powered hydraulic pump and adjustment of the flow control valves. These components were discussed in Section 4.2.

During the initial static and final partially restrained-head tests, the rate varied because of the continuous cycles of loading and unloading (sawtooth behavior). For the cyclic tests, attempts were made to keep the loading rate at about 1 minute per cycle. The actual loading rate for the computer-controlled mode varied from 1 to 8 minutes per cycle. The longer duration occurred for tests at high load levels where the loading system was operating near its capacity.

5.5 Observation of Soil Deformation Near Pile Surface

Except for the initial PRH test, the area around the test piles was flooded by sea water during testing. The flooding of the test site was ideal because it assured that the soil deposit was saturated during testing. Since the surficial soil was submerged, observation of soil deformation at the ground surface was limited to periods of low tides when the surficial soil adjacent to the test piles was exposed.

The large displacement cyclic tests were performed on the last day of testing and the test site was flooded during pile loading. Observation of soil deformation after the cyclic tests were carried out early next day when the high tides had subsided.

The most noticeable soil deformation pattern was the formation of a conical soil depression zone around the test piles. The diameter of the depression extended about 1 to 2 feet (0.3 to 0.6 m) away from the pile surface. The cone-shaped soil deformation pattern was probably caused by a combination of soil compaction, subsidence, and possibly by scour during cyclic loading.

TEST SERIES	PILE HEAD CONDITION	START DATE AND TIME	END DATE AND TIME	TYPE	REMARKS
1	PARTIALLY RESTRAINED	22 SEPT 1738	22 SEPT 2002	STATIC	DEBUG TEST AND DATA ACQUISITION SYSTEM
2 TO 6	FREE	23 SEPT 0854	24 SEPT 1436	CYCLIC	5 TESTS WERE PERFORMED NO TEST BETWEEN 1900 OF 23RD AND 0723 OF 24TH
6 TO 9	FULLY RESTRAINED	24 SEPT 1511	24 SEPT 1811	CYCLIC	3 TESTS WERE PERFORMED
10	PARTIALLY RESTRAINED	24 SEPT 1843	24 SEPT 2014	STATIC	LOADING IN SMALL INCREMENTS

TABLE 5-1 SUMMARY OF TEST PROGRAM

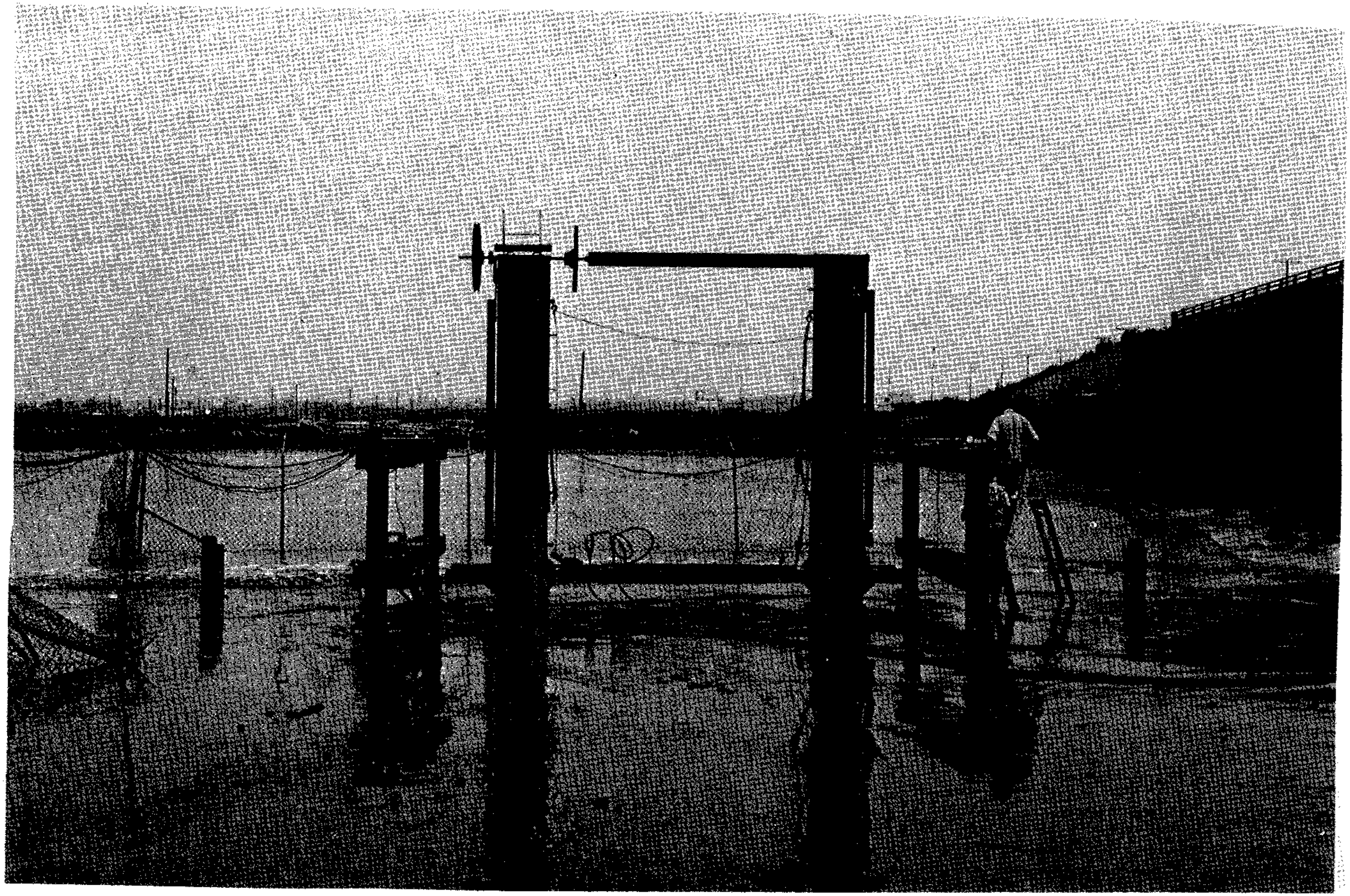
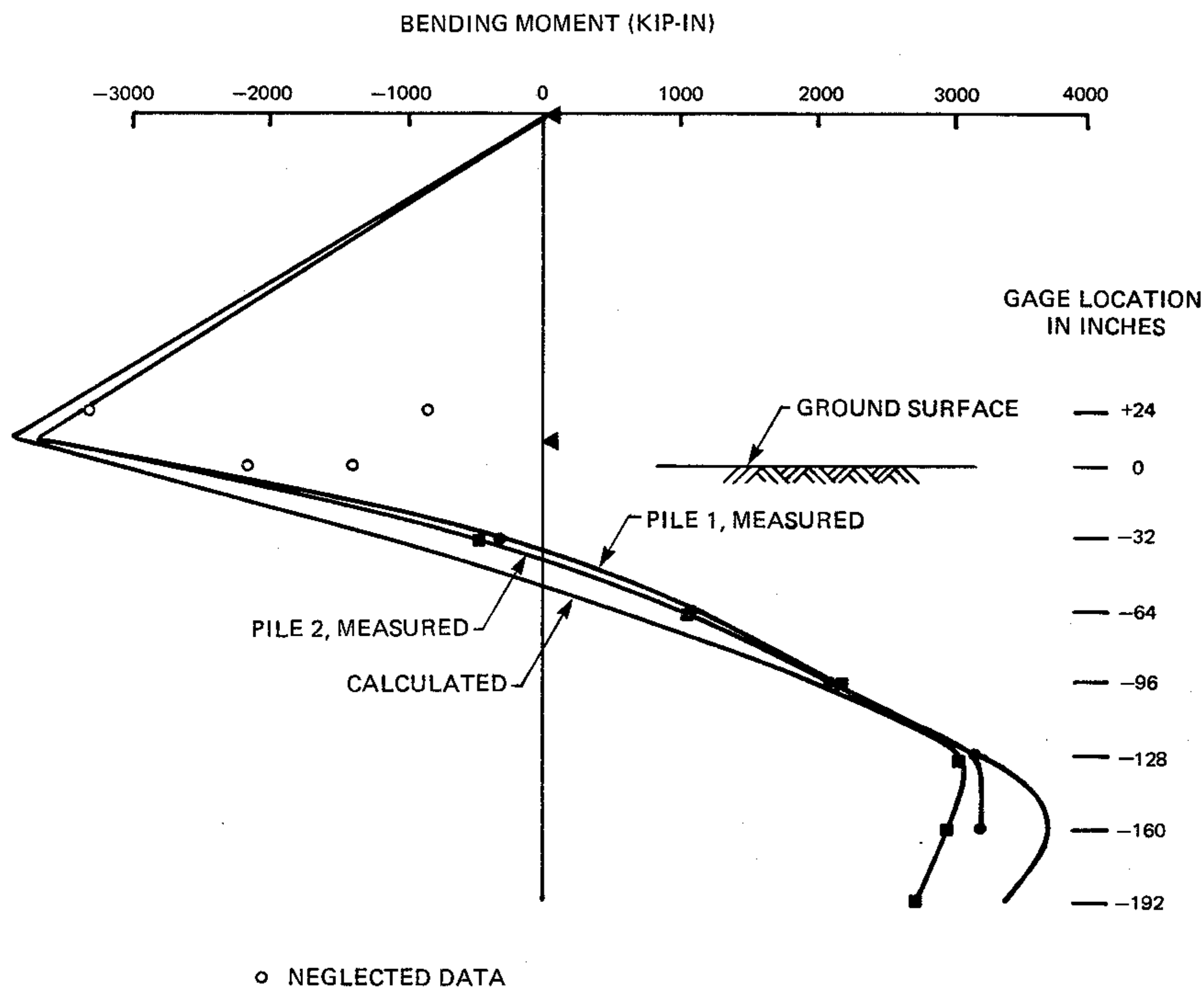


FIGURE 5-1 ACTUAL TEST SET-UP (COMPARE TO FIGURE 4-1)



Ertec
The Earth Technology Corporation

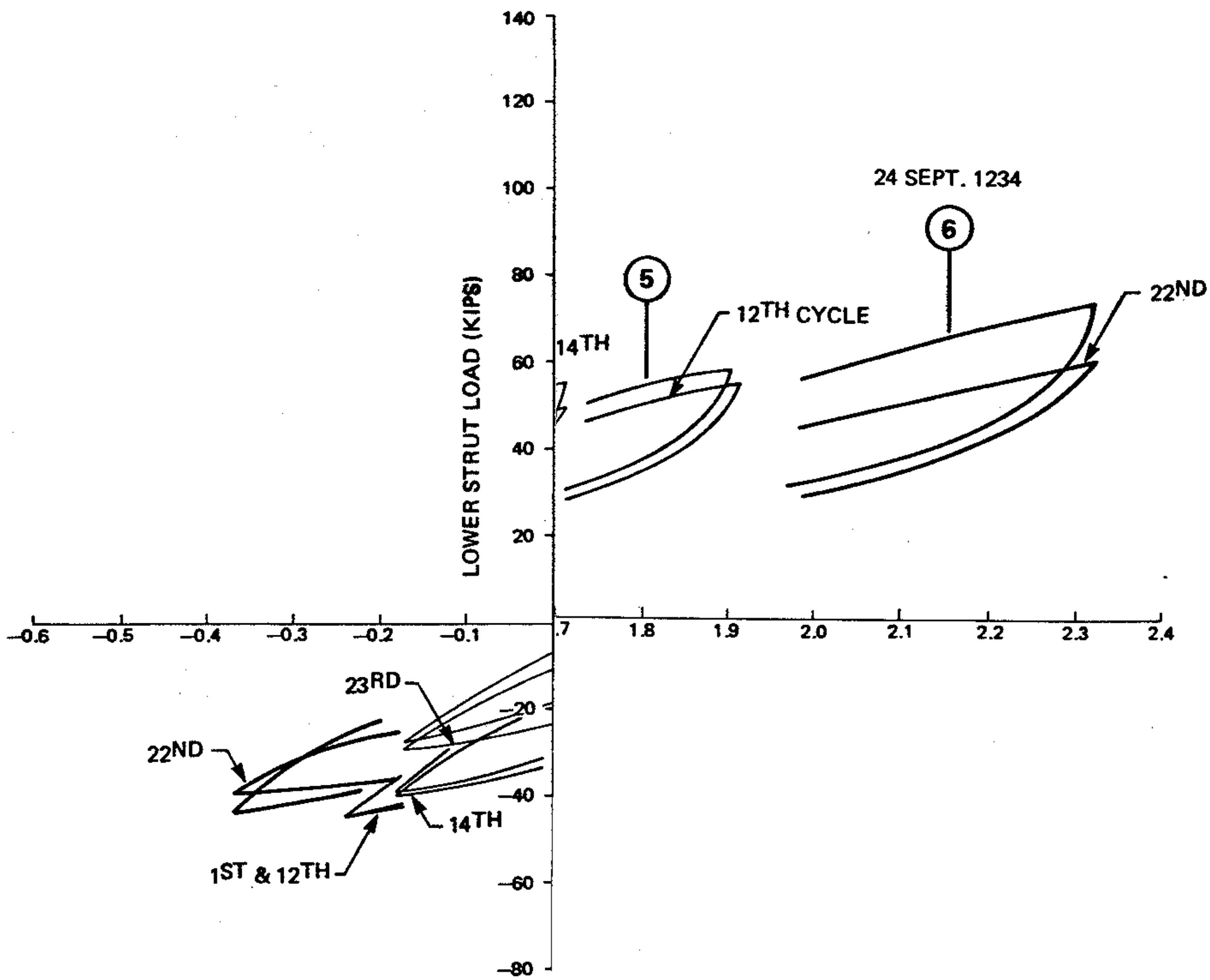
PROJECT NO.: 82-205

SEAL BEACH

CALCULATED VERSUS MEASURED BENDING MOMENT DISTRIBUTIONS AT POINT C OF FINAL PARTIALLY RESTRAINED-HEAD TEST

8-83

FIGURE 7-9



Ertec
The Earth Technology Corporation

PROJECT NO.: 82-205

SEAL BEACH

SUMMARY PLOT OF LOAD VERSUS DEFLECTION FOR COMPLETE TEST PROGRAM

8-83

FIGURE 5-2

6.0 PRESENTATION OF TEST RESULTS

6.1 Measurements

A review of the types of data recorded during the field tests is beneficial to support the discussions in this chapter. The two test piles were instrumented to provide bending moment measurements to a depth of 16 feet (4.9 m) below the ground surface. Each pile-head response was monitored by measured loads and deflections. Piezometers, installed at a depth of 9.8 ft (3.0 m) below the ground surface, were intended to measure pore pressure fluctuations during testing. Pile inclination was also periodically recorded.

6.2 Pile Head Behavior

A fundamental requirement for all pile load tests is to monitor load-deflection relationships at the pile head. For the Seal Beach tests, this relationship was displayed instantaneously on an X-Y plotter during loading to facilitate control of the tests. A complete set of the lower strut load-deflection plots are presented in the following section. The deflection values were taken from Pile 2. The zero readings used for all sets of plots were taken before initiation of the first test.

Partially Restrained-Head (PRH) Test. Two series of PRH tests were performed. The piles were tested by applying a load increment at the bottom load strut and then adjusting the threaded bearing wheel at the top load strut to achieve a prescribed ratio of lateral deflection at the two strut levels. This deflection ratio, as prescribed in Chapter 2, was about 1.4. The actual ratio for the first PRH test was back-calculated and varied from 1.1 to 1.7. This discrepancy was due to rapid pile relaxation and lack of precise control of deflection at the pile top. The "deflection ratio" procedure was subsequently changed for the final PRH test. The loading was performed in very small increments of deflection in the final PRH test.

The load-deflection relationships for the two PRH tests are presented in Figs. 6-1 and 6-2. In Fig. 6-1, a sawtooth type behavior resulted from the loading procedure described earlier. The maximum lower strut load and deflection were about 80 kips (356 kN) and 0.66 inch (1.7 cm), respectively. A permanent set of about 0.3 inch (0.8 cm) was also measured at the lower strut level at the end of the test. This permanent set, which remained stable throughout the subsequent tests, was thought to be caused by the falling-in of soil at the back of the pile during initial loading (grain migration) and subsequent compaction of the soil when the pile unloaded in the opposite direction. For piles embedded in sands, the behavior is biased in the direction of first loading. This means that the lateral response of a pile is affected by the loading history of the virgin test (Matlock and Ripperger, 1957).

A similar test was performed at the end of the program. The result for this final PRH test is presented in Fig. 6-2. As shown in Fig. 6-2, the sawtooth type response is less pronounced at the beginning of the test because loading was done in very small increments of deflection. As the test approached higher load levels, it became more difficult to control the rotation of the threaded bearing wheel; as a result, the sawtooth behavior was again recorded. A peak lower strut load of about 90 kips (400 kN) was measured at a deflection of 1.4 inches (3.6 cm).

Free-Head (FH) Tests. Five series of FH tests were performed. Incremental deflection limits used for this test were

(1) 0 and 0.9 inch (0 and 2.3 cm),

(2) -0.09 and 1.4 inches (-0.3 and 3.6 cm),

(3) -0.11 and 1.6 inches (-0.3 and 4.1 cm),

(4) -0.2 and 1.7 inches (-0.5 and 4.3 cm), and

(5) -0.3 and 2.1 inches (-0.8 and 5.3 cm).

These sets of deflection limits, imposed at the lower strut, were selected on the basis of the 3:1 ratio of major versus minor deflections discussed in Chapter 5. The total number of cycles performed for each of the five tests were 7, 23, 14, 12 and 22, respectively. The loading rate varied from about 1 to 8 minutes per cycle.

A simplified presentation of the load-deflection behavior for the FH tests is shown in Fig. 6-3. Only the first cycle of each FH test is presented in Fig. 6-3. Data for the subsequent cycles were available from the field analog plots used for test observation and control. The load-deflection coordinate before each reversal of loading was also recorded by the computer. Although these data points and analog plots could be included in Fig. 6-3, they are omitted to prevent congestion.

In addition to the partial hysteresis loops, Fig. 6-3 also shows the amount of load reduction at the end of cycling for each test. A more detailed interpretation of soil degradation is presented in the next chapter. A careful study of Fig. 6-3 indicated that a fairly smooth curve could be fitted through the degraded levels.

<u>Fully Restrained-Head (FRH) Tests</u>. Although the surficial soil was subjected to large deformation and possible cyclic resistance degradation during the FH tests, the FRH tests were performed as part of the test program to investigate the effects of boundary conditions on the measured load-deflection relationship and bending moment distribution. Three FRH tests were performed. The deflection limits in the minor and major directions were

(1) 0.22 and 0.52 inch (0.6 and 1.3 cm),

(2) 0.14 and 0.75 inch (0.4 and 1.9 cm), and

(3) 0.05 and 1.01 inches (0.1 and 2.6 cm).

The total number of cycles for each FRH test were 7, 7 and 12. The loading rate was about 2 minutes per cycle. A simplified plot of the measured load-deflection behavior for the three FRH tests is shown in Fig. 6-4.

Similar to the FH test results, only the first cycle of each FRH test is presented in Fig. 6-4. As reflected by the change in head restraint, the stiffness of the load-deflection curve increased significantly as compared to the FH tests. Unlike the FH tests, soil resistance variation which occurred during cyclic testing was minimal. For the first and second FRH tests, a small increase, rather than reduction, was observed in soil resistance due to cyclic loading.

6.3 Bending Moment Distribution

Bending moment distribution along the pile length is one of the most important parameters considered in lateral pile design. Bending moment comparisons were also used in this study as the key method of evaluating p-y criteria. The magnitude and location of maximum bending moment along the pile length is influenced by the pile-head boundary condition as well as the type of lateral loading (static or cyclic). In this study, the bending moment distribution for the top 16 feet (4.9 m) below the ground surface was measured under three different boundary conditions for both static and cyclic loadings. A fixed set of zero readings were used to generate the bending moment distributions. This set of readings was taken before the first load test.

Measurements taken at the first and second strain gage stations were frequently discarded in these tests because of the proximity of these strain gage stations to the point of load application. Stress concentration and local deformation of the pile material will influence these strain gage outputs, resulting in erroneous readings. The locations of the strain gage stations were determined in the earlier vibration tests where loading was applied at the pile top. So the bending moment at the point of load application (lower strut) was calculated by the product of the measured top strut load and the distance between the two load struts.

With the exception of the eighth (last) station of Pile 1 which was malfunctioning in the earlier NSF tests, all the instruments functioned properly. This was confirmed by stable manual balance box readings taken periodically throughout the test program and a high resistance to ground measured before and several months after the load test. The resistance to ground for the malfunctioning strain gage station was only 4 kΩ, indicating poor isolation from ground. The measured data for this strain gage station were thus disregarded for all tests.

Partially Restrained-Head (PRH) Tests. As mentioned earlier, two PRH tests were performed for this study. The first was the virgin static test and the second was the last test performed in this study. A set of selected bending moment distributions for Piles 1 and 2 of the virgin test are presented in Figs. 6-5 and 6-6, respectively. The circled numbers at the bottom of each curve are used for cross referencing with the load-deflection plot of Fig. 6-1. Comparing the contents of Figs. 6-5 and 6-6 with Fig. 6-1 will identify the load increments at which the moment data were sampled. The bending moment at the point of load application (lower strut) was calculated based on the measured top strut load and the distance between the two load struts. These computed moments are plotted on Figs. 6-5 and 6-6. For the reasons previously explained, data points close to the point of loading were often neglected and are shown in these figures as small open circles.

The measured moment distributions from these series of tests indicated that the boundary constraint was varying between a restrained and almost free-head condition. This resulted in the shifting of the absolute maximum bending moment from the point of load application to a depth of 6 to 7 times the pile diameter below the ground surface. The location of the maximum positive (at-depth) bending moment also was varying because of nonlinear soil behavior.

The curves represented by dashed lines in Figs. 6-5 and 6-6 were recorded at the end of the first increment of unloading. The reversal in pile curvature increased slightly the at-depth bending moment and also reduced the bending moment at the lower load strut level.

The results of the final PRH test are shown in Figs. 6-7 and 6-8. Again, only a selected set of the measured bending moment distributions is presented. The load increment at which these bending moments were recorded can be found by comparing Fig. 6-2 with Figs. 6-7 and 6-8. Similar to the virgin PRH test, the bending moment at the lower load strut level was calculated by the product of the measured top strut load and the distance between the two load struts.

The distribution of bending moment along the pile length are quite similar to that observed in the virgin test. However, some discontinuities, which were not evident in the virgin test, were clearly shown at deeper points. These discontinuities may be due to some degree of local yielding of the pile material which occurred at high stresses during the earlier Fully Restrained-Head Test to be discussed later.

Free-Head (FH) Tests. Selected plots of the bending moment distributions for this series of tests are shown in Figs. 6-9 and 6-10 for Piles 1 and 2, respectively. These data points were recorded at the peak load of the first cycle of each FH test.

Under this boundary condition, the maximum bending moment occurred consistently at depths of about 11 feet (3.4 m) or 5.5 times the pile diameter below the ground surface. An additional bending moment curve is also shown in Figs. 6-9 and 6-10 to illustrate the effect of cyclic loading. As shown in the figures, cyclic loading under imposed deflection limits tends to relax the bending moment along the pile length.

Fully Restrained-Head (FRH) Tests. Three FRH tests were performed for this study. The bending moment distributions recorded at the peak load of the first cycle are shown in Figs. 6-11 and 6-12 for Piles 1 and 2, respectively. The maximum bending moment at the lower load strut was again computed based on the measured top strut load and the distance between the two load struts. For the last FRH test, a maximum bending moment of about 8000 kip-in (904 kN-m) was recorded. This magnitude of bending moment is equivalent to a bending stress of 34 ksi (234,000 kPa), which was a "near-yield" condition for the pile material. The effect of cyclic loading under this type of boundary condition was negligible.

6.4 Pore Water Pressure Response

During testing, the visually observed standpipe piezometers (Fig. 4-1) were not used to monitor the pore water pressure response because the electrical pore pressure transducer unit located closer to the pile wall was already registering no pore water pressure variation during testing. This transducer was installed at one foot (0.3 m) away from the pile wall at a final penetration of 9.8 feet (3.0 m), which was immediately below the silty clay layer.

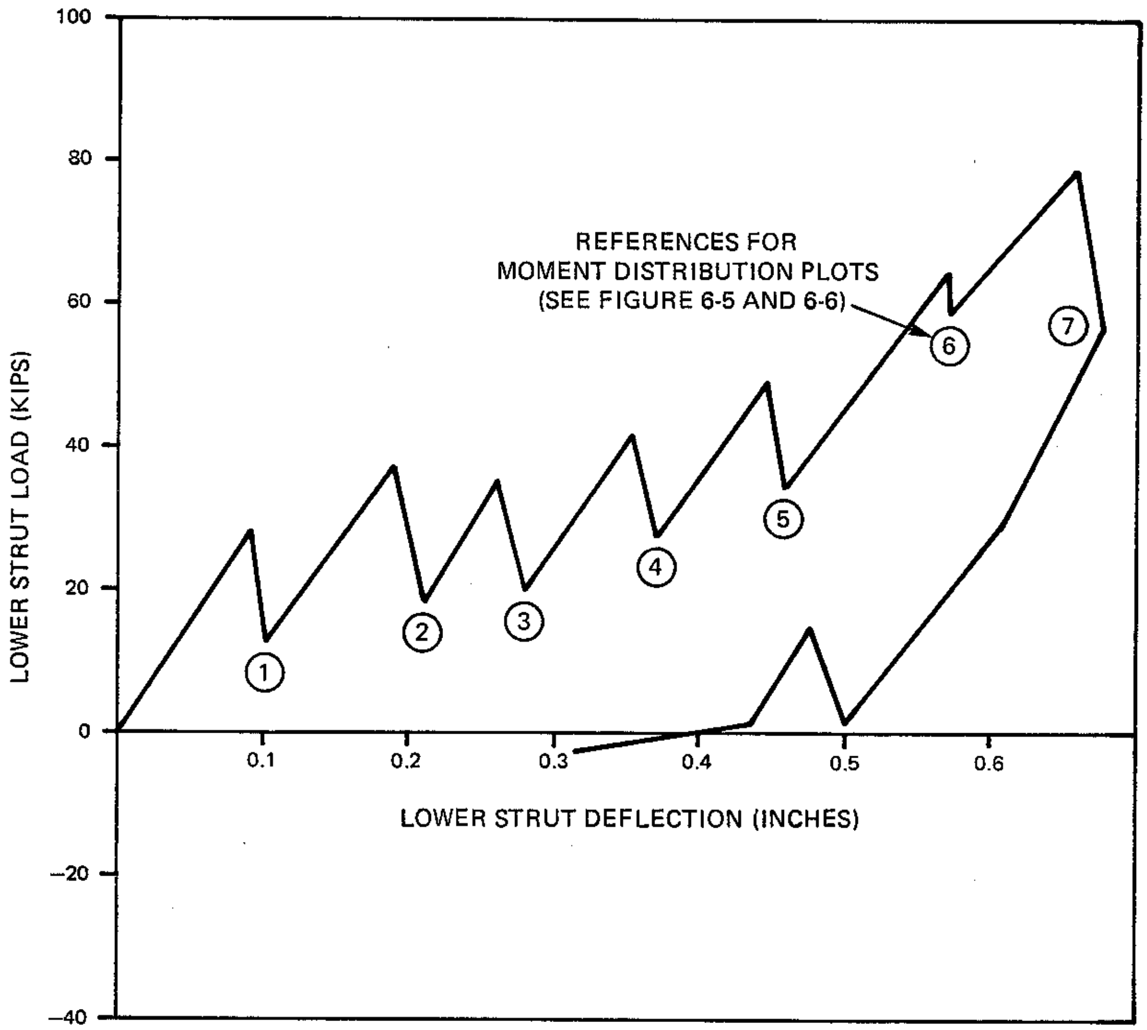
The maximum rate of loading during cyclic testing was about 1.0 minute per cycle. This slow loading rate, together with the high permeability of the soil and the shallow penetration of the porous piezometer tip, resulted in no observable change of pore water pressure during testing. The observed changes in soil resistance during cycling may therefore be considered to be primarily mechanical volume changes and movements of the soils.

6.5 Pile Inclination Measurements

Slope measurements were taken at the beginning and end of each test at a zero load condition. These measurements were used to detect any permanent tilting of the piles after testing.

In general, the micrometer readings indicated a significant change in slope after the initial PRH test. Thereafter, the slope before and after each test remained about the same. The slope after the initial PRH test was measured to be 0.0006 radian and 0.0014 radian for Piles 1 and 2, respectively. These values corresponded to a change of -0.0017 radian and +0.0005 radian for the two piles before and after the initial PRH test. For the remaining tests, the before and after-test readings varied from 0.0003 to 0.0005 for Pile 1 and 0.0013 to 0.0015 radian for Pile 2.

The large variation in pile inclination for the initial PRH test was also reflected in the deflection measurements taken before and after the test. A permanent deflection of about 0.3 inch (0.8 cm) was recorded at the lower strut level for the initial PRH test and no permanent translation of the test piles were observed for the remaining tests. As discussed in Section 6.2, this permanent set was probably caused by grain migration during loading and subsequent soil compaction upon pile unloading.



Ertec
The Earth Technology Corporation

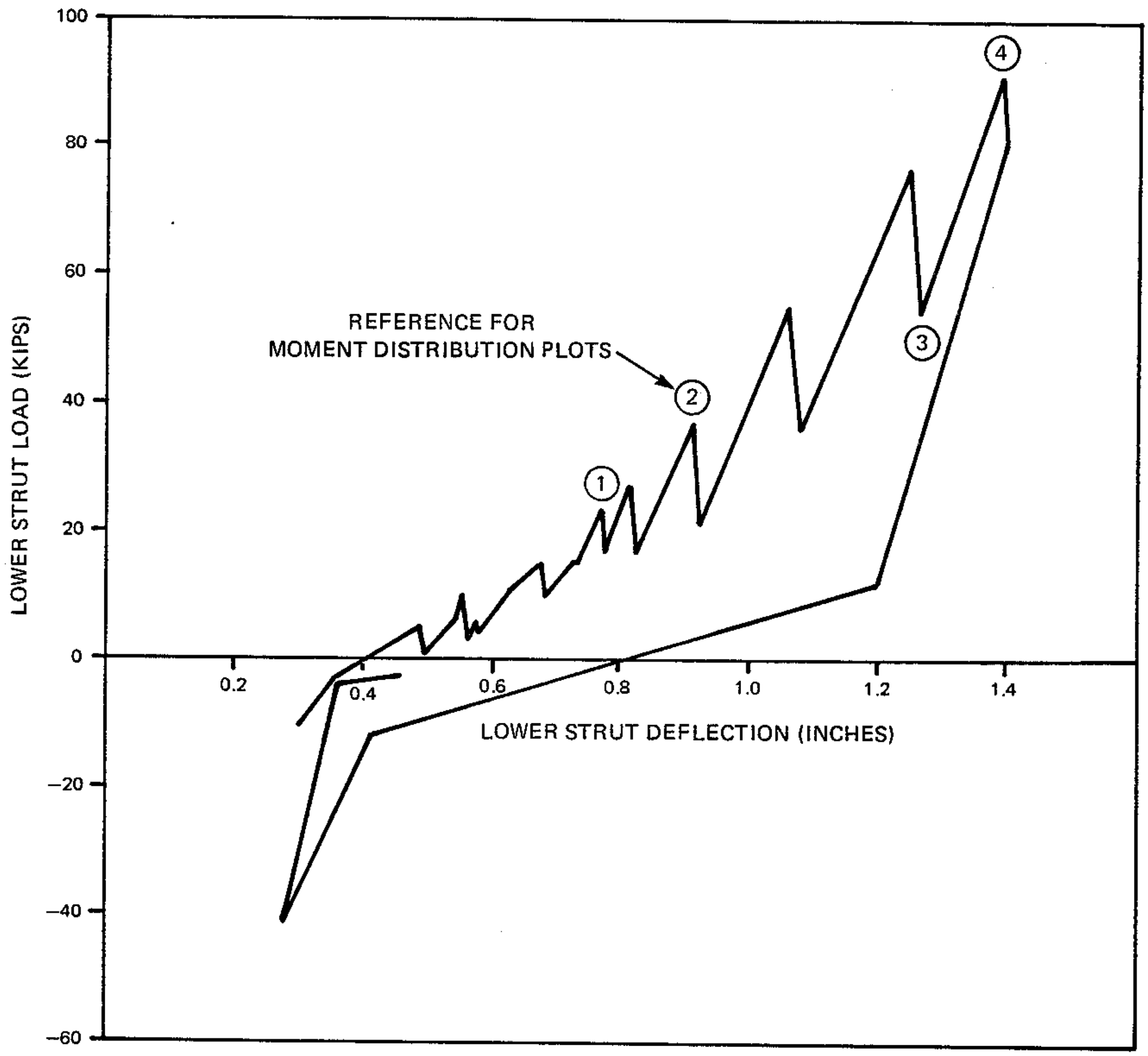
PROJECT NO.: 82-205

SEAL BEACH

LOWER STRUT LOAD-DEFLECTION RELATIONSHIP FOR INITIAL PARTIALLY RESTRAINED-HEAD TEST

8-83

FIGURE 6-1



Ertec
The Earth Technology Corporation

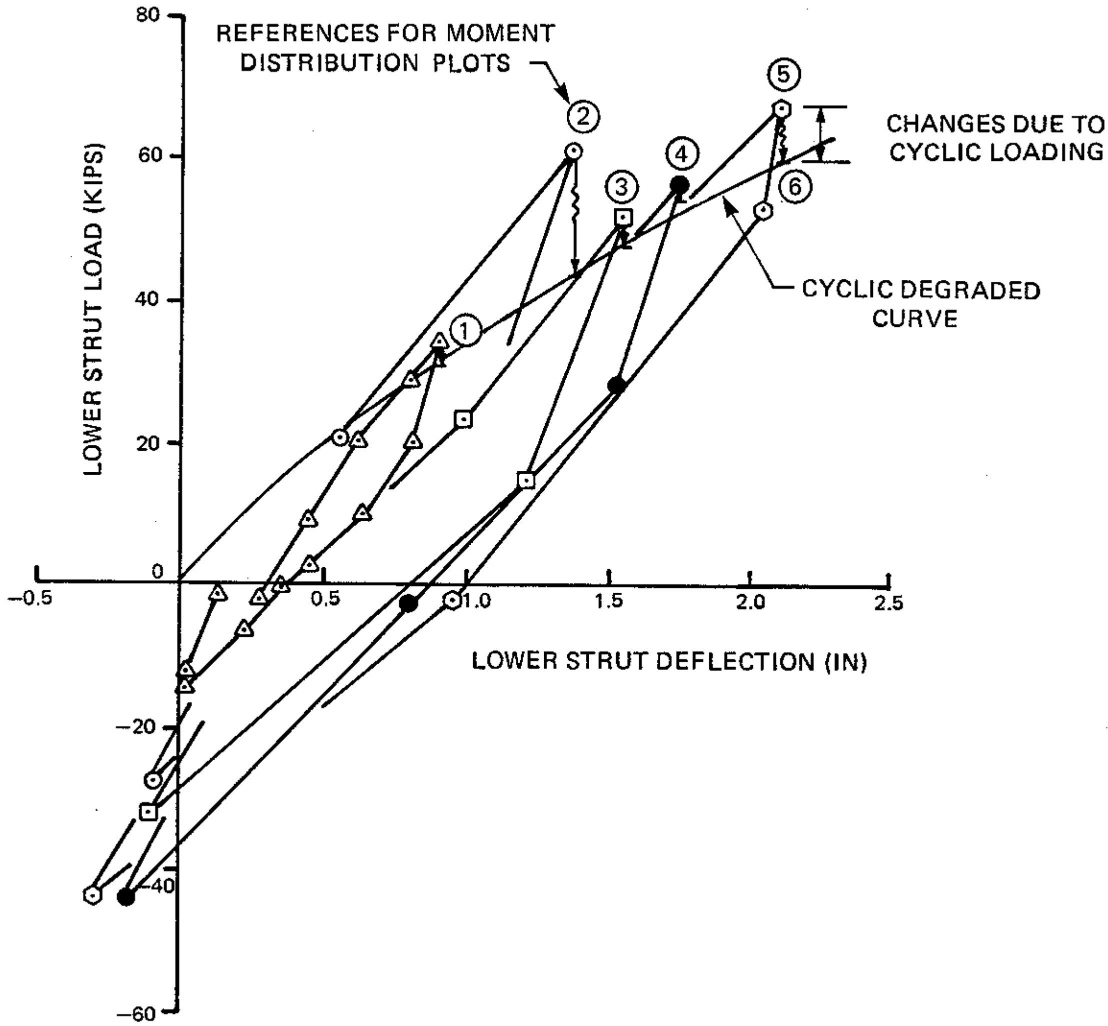
PROJECT NO.: 82-205

SEAL BEACH

LOWER STRUT LOAD-DEFLECTION RELATIONSHIP FOR FINAL PARTIALLY RESTRAINED-HEAD TEST

8-83

FIGURE 6-2



Ertec
The Earth Technology Corporation

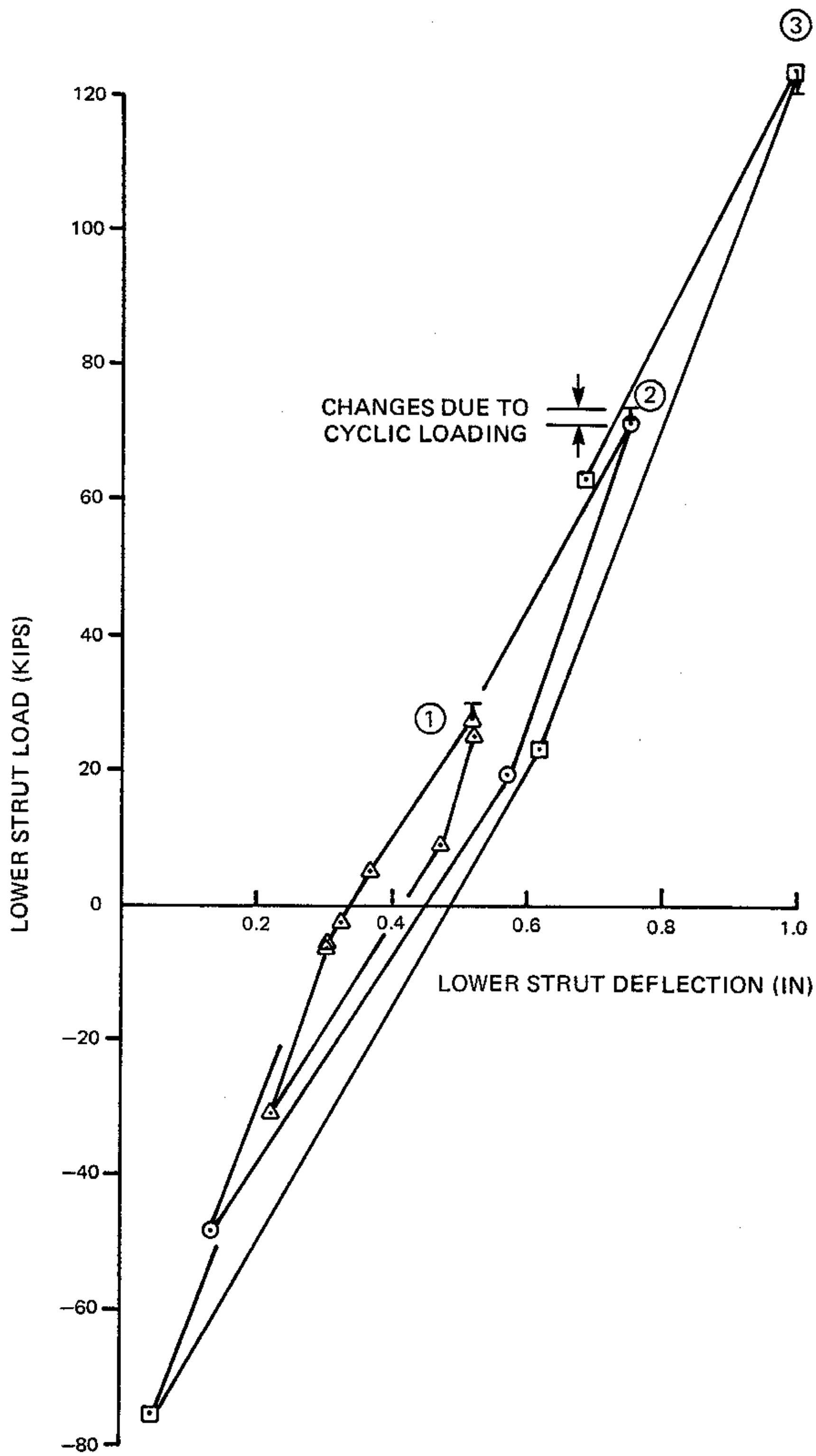
PROJECT NO.: 82-205

SEAL BEACH

LOWER STRUT LOAD-DEFLECTION RELATIONSHIP FOR FIVE FREE-HEAD TESTS

8-83

FIGURE 6-3



Ertec
The Earth Technology Corporation

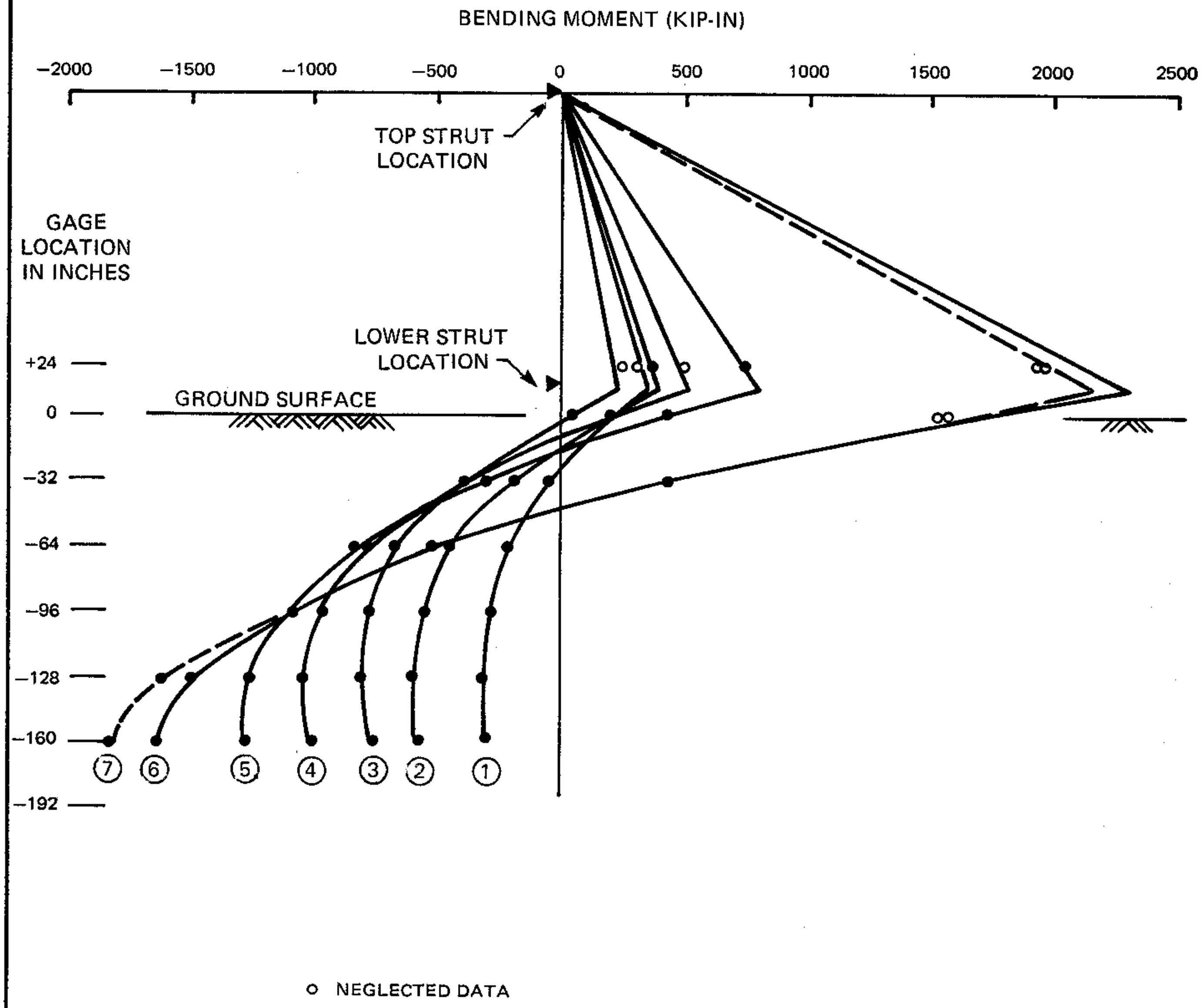
PROJECT NO.: 82-205

SEAL BEACH

LOWER STRUT LOAD-DEFLECTION RELATIONSHIP FOR THREE FULLY RESTRAINED-HEAD TESTS

8-83

FIGURE 6-4



Ertec
The Earth Technology Corporation

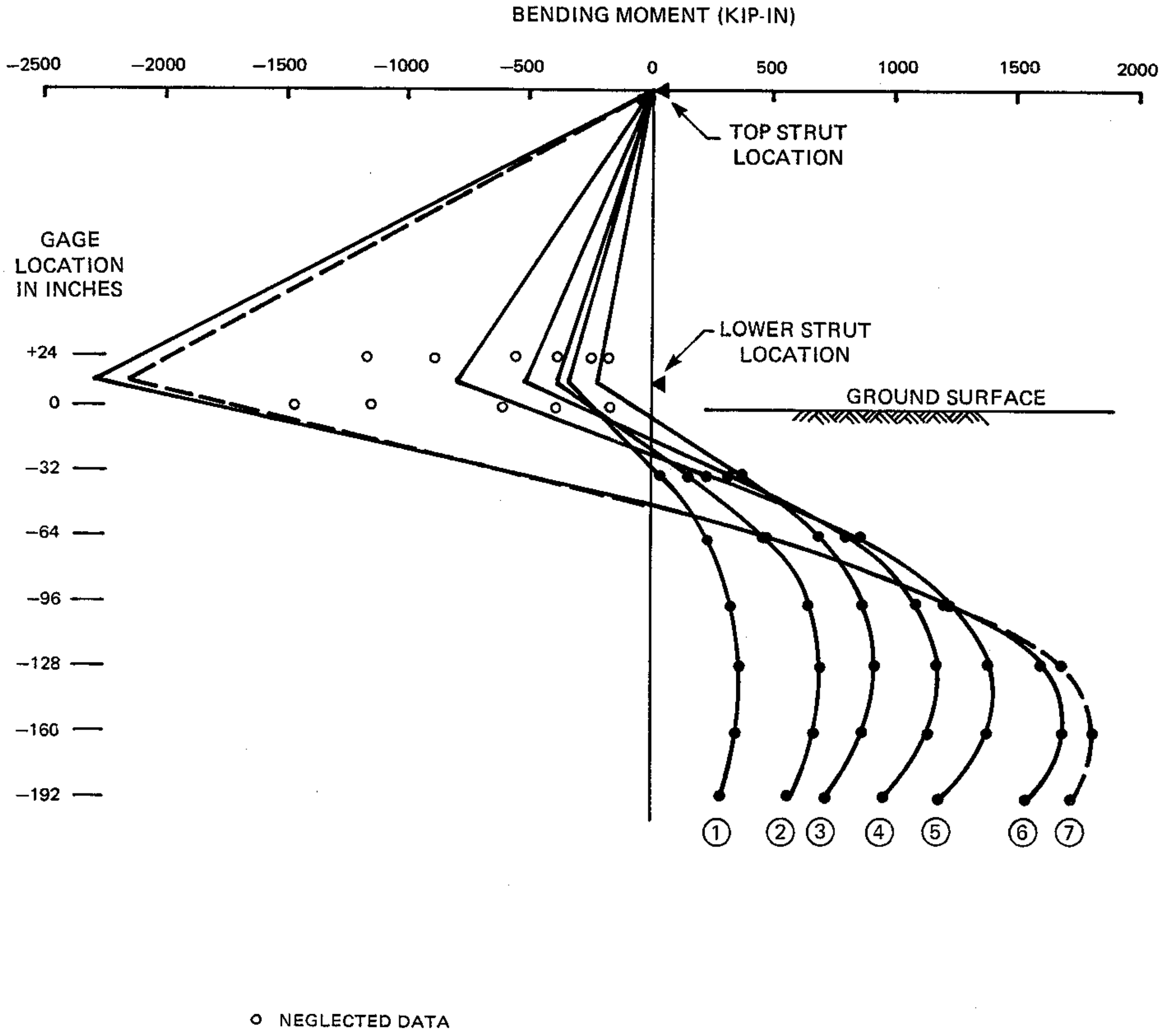
PROJECT NO.: 82-205

SEAL BEACH

BENDING MOMENT DISTRIBUTIONS MEASURED BY PILE 1 DURING INITIAL PARTIALLY RESTRAINED-HEAD TEST

8-83

FIGURE 6-5



Ertec
The Earth Technology Corporation

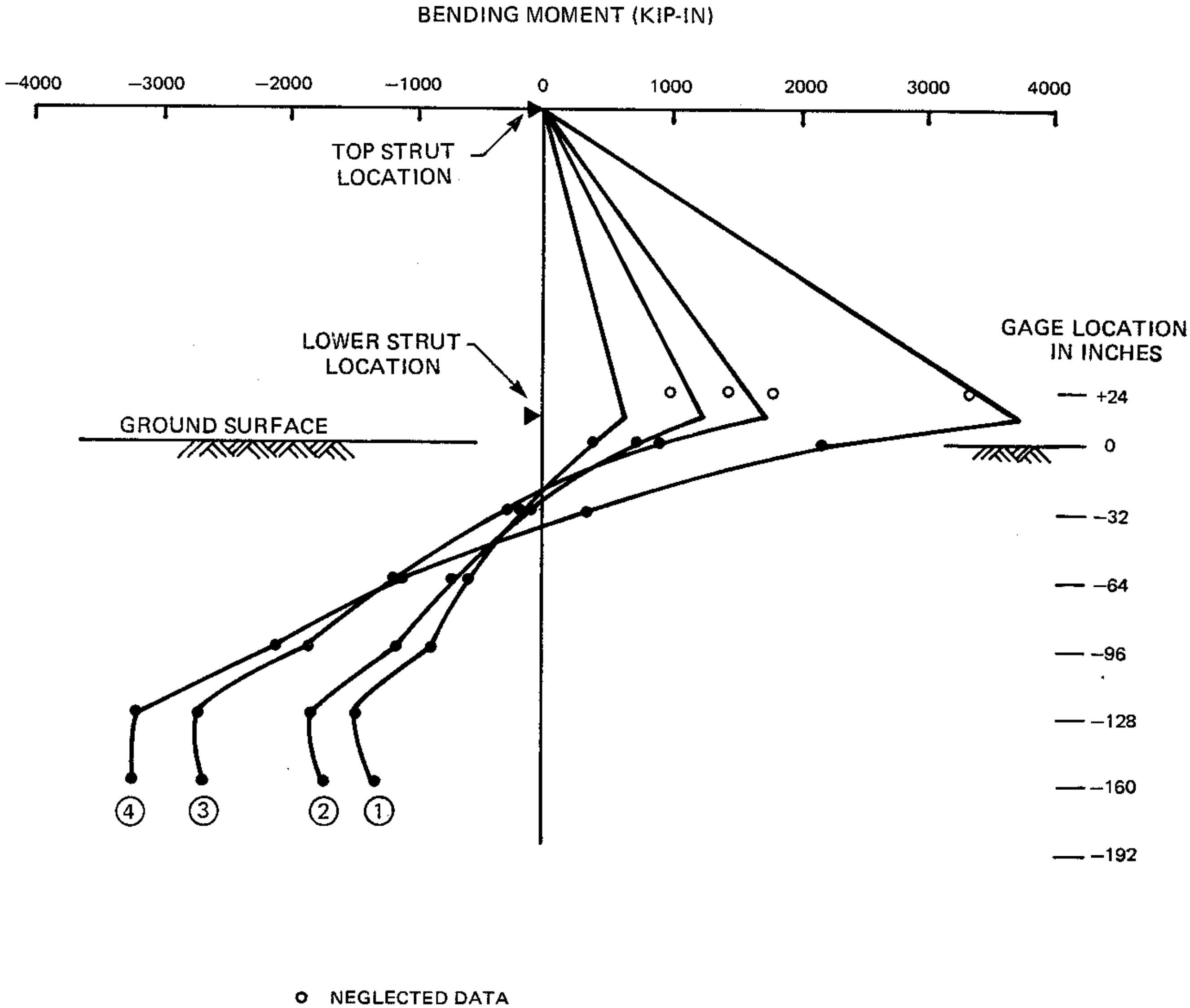
PROJECT NO.: 82-205

SEAL BEACH

BENDING MOMENT DISTRIBUTIONS MEASURED BY PILE 2 DURING INITIAL PARTIALLY RESTRAINED-HEAD TEST

8-83

FIGURE 6-6



Ertec
The Earth Technology Corporation

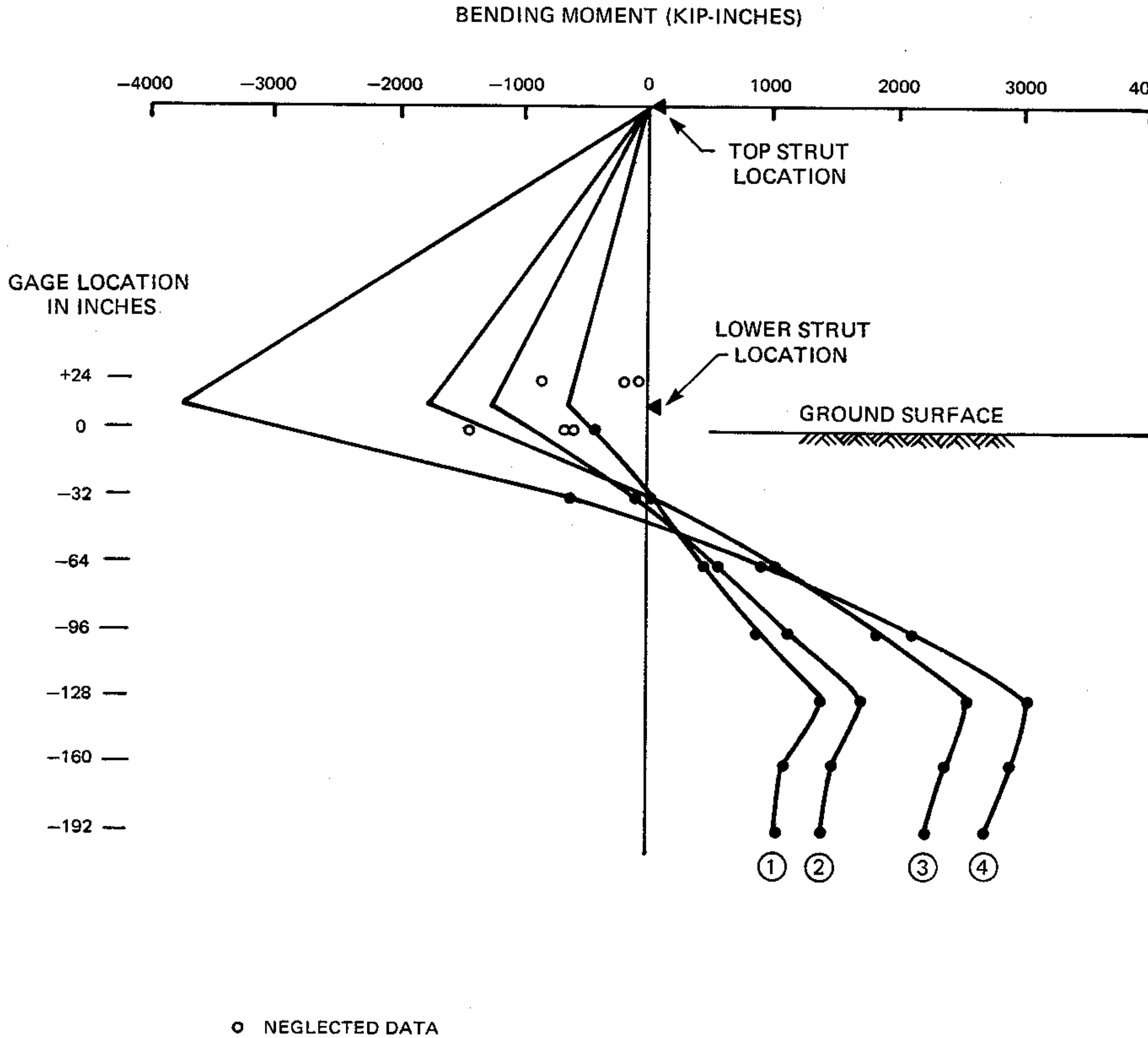
PROJECT NO.: 82-205

SEAL BEACH

BENDING MOMENT DISTRIBUTIONS MEASURED BY PILE 1 DURING FINAL PARTIALLY RESTRAINED-HEAD TEST

8-83

FIGURE 6-7



Ertec
The Earth Technology Corporation

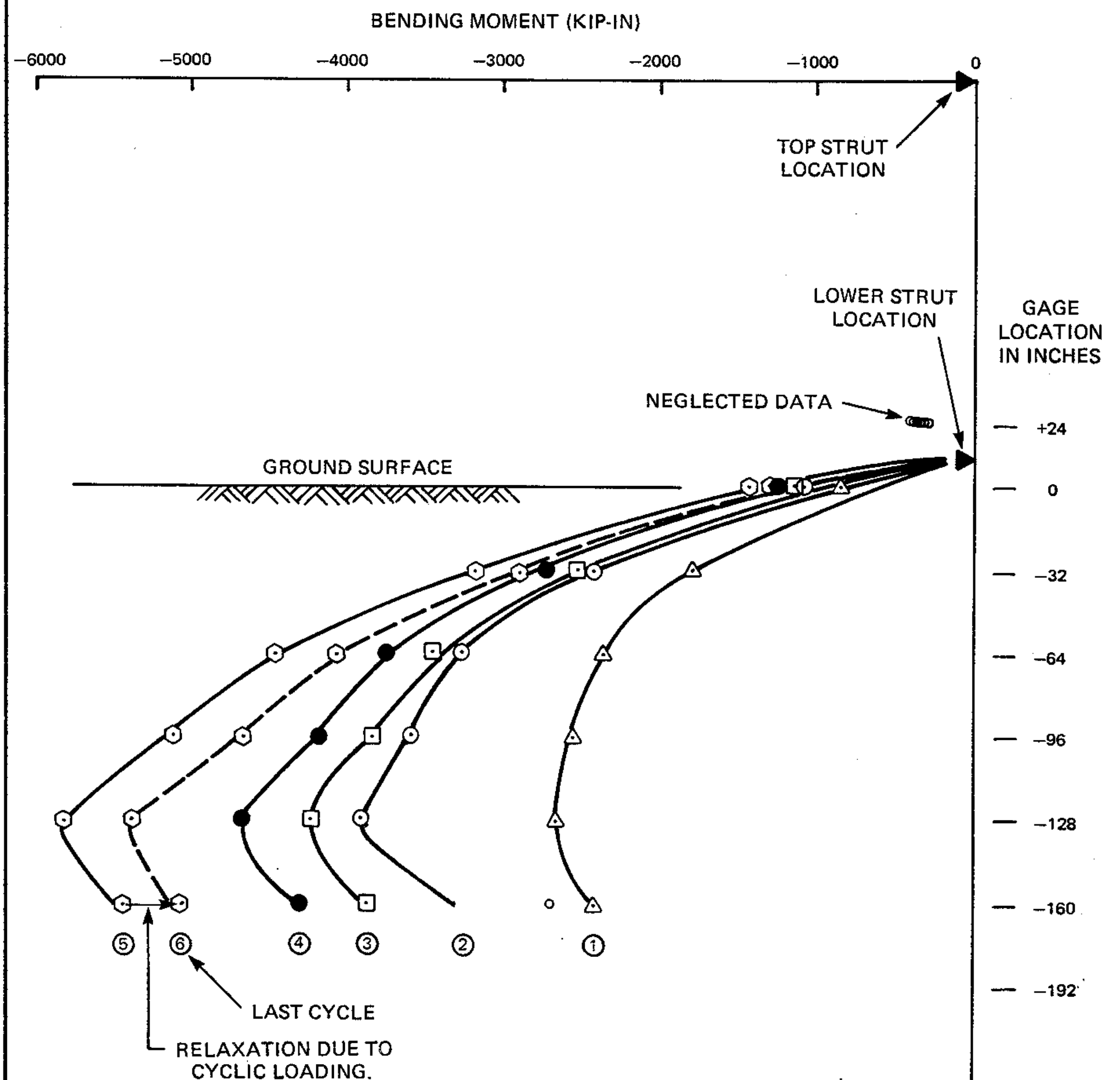
PROJECT NO.: 82-205

SEAL BEACH

BENDING MOMENT DISTRIBUTIONS MEASURED BY PILE 2 DURING FINAL PARTIALLY RESTRAINED-HEAD TEST

8-83

FIGURE 6-8



Ertec
The Earth Technology Corporation

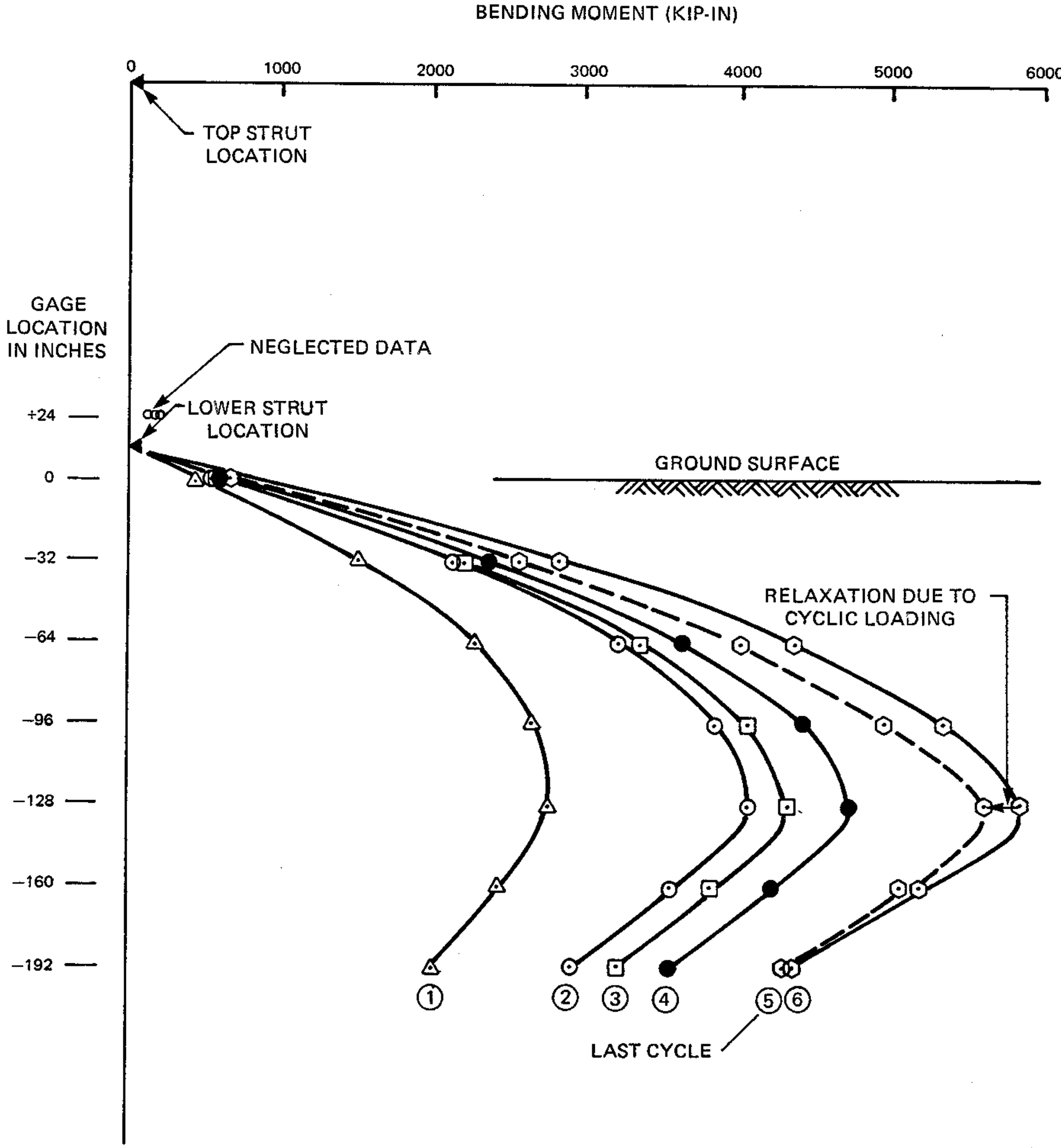
PROJECT NO.: 82-205

SEAL BEACH

BENDING MOMENT DISTRIBUTIONS
MEASURED BY PILE 1
DURING FREE-HEAD TEST

8-83

FIGURE 6-9



Ertec
The Earth Technology Corporation

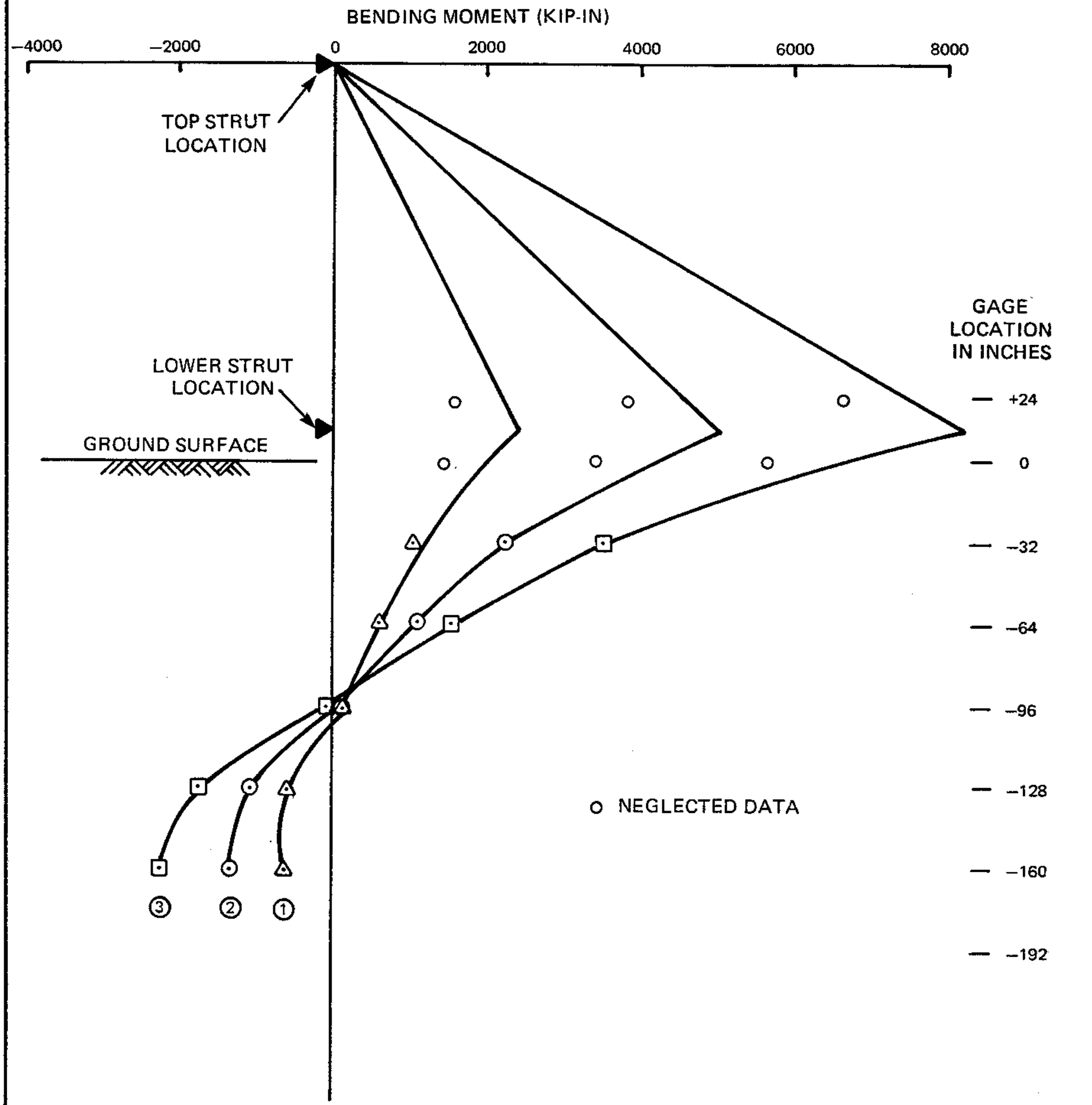
PROJECT NO.: 82-205

SEAL BEACH

BENDING MOMENT DISTRIBUTIONS
MEASURED BY PILE 2
DURING FREE - HEAD TEST

8-83

FIGURE 6-10



Ertec
The Earth Technology Corporation

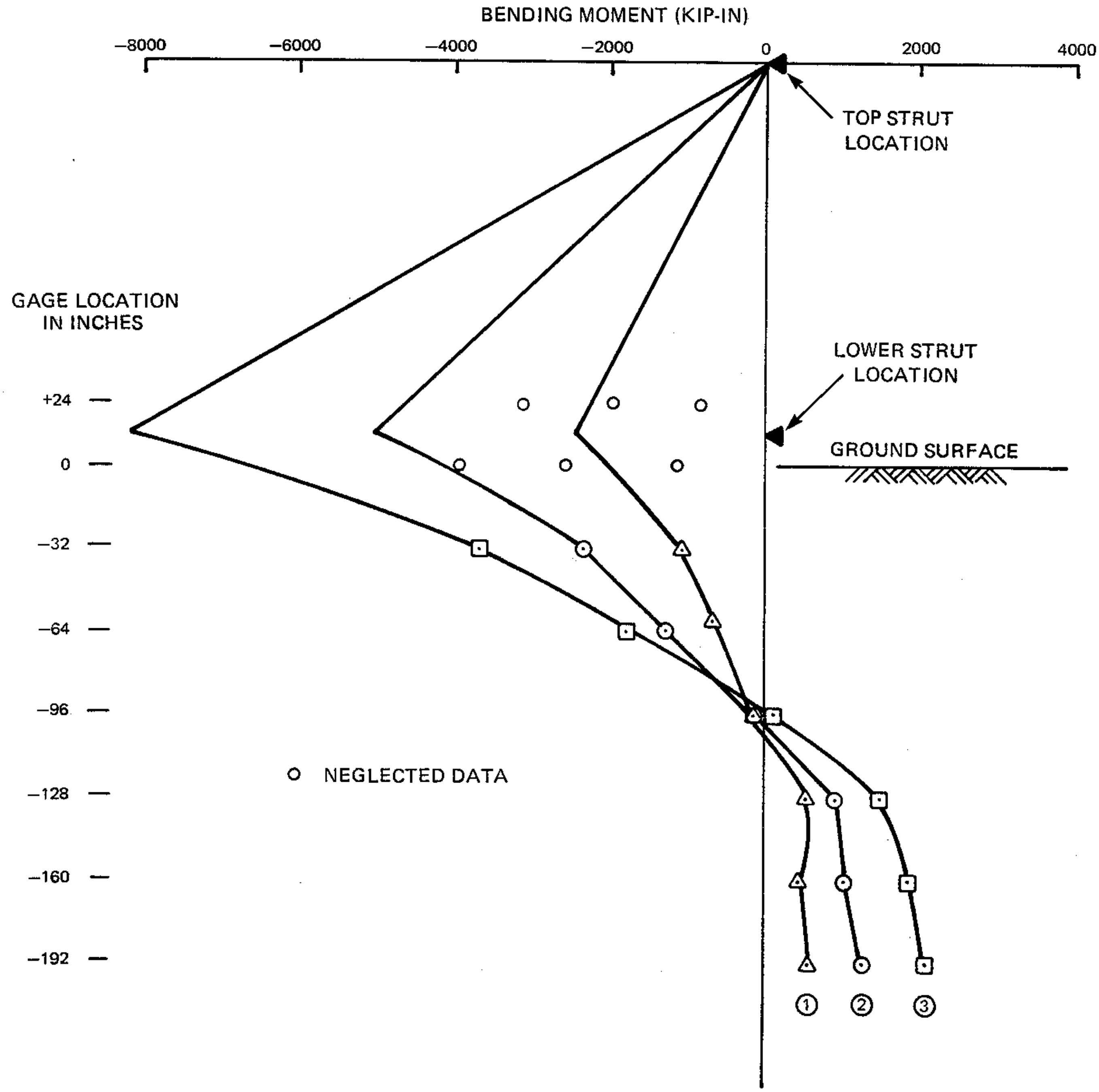
PROJECT NO.: 82-205

SEAL BEACH

BENDING MOMENT DISTRIBUTIONS MEASURED BY PILE 1 DURING FULLY RESTRAINED-HEAD TEST

8-83

FIGURE 6-11



Ertec
The Earth Technology Corporation

PROJECT NO.: 82-205

SEAL BEACH

BENDING MOMENT DISTRIBUTIONS MEASURED BY PILE 2 DURING FULLY RESTRAINED-HEAD TEST

8-83

FIGURE 6-12

7.0 ANALYSIS OF TEST RESULTS

7.1 General

With the limited number of moment data points along the pile, direct differentiation and integration to obtain points on p-y curves was not possible for this project. Instead, a procedure was considered to backfit the measured bending moment curves by assuming some form of soil modulus variation with depth and incorporating it into a pile solution. By adjusting to the actual data, this procedure would eventually yield an approximate family of p-y curves which would be used for comparison with the recommendations given by API (1982). This type of post-analysis was altered when the silty clay layer at the test site was found. The presence of this clay layer led to a trade-off in favor of the study of an ideal layered soil system.

To adapt the field measurements for comparison with established design methods, an alternate post-analysis procedure was used. Based on the results of the site investigation and laboratory testing program, a set of soil parameters was selected to define the p-y relationships for the layered soil. These p-y curves were used in a beam-column analysis to compute the pile-head load-deflection relationships and bending moments along the pile. Then, the results of the API procedure were evaluated by comparing the calculated results with the field measurements.

Soil degradation during the cyclic tests was also evaluated using pile-head measurements. The effect of boundary conditions and the rate of loading was also studied. Finally, a comparison of LVDT deflections was made between Piles 1 and 2 for possible variation in behavior due to earlier vibration of Pile 1 and differences in soil condition.

7.2 Analytical Tool

The computer program, SPASM (Matlock et al, 1979) was used to analyze the test data. Discrete-element mechanical analogs are used to represent the pile-soil system in SPASM. The pile is a linearly elastic beam and is modelled by a series of lumped masses and springs. The soil support model consists of an assemblage of elasto-plastic subelements. A mechanistic view of this representation is shown in Fig. 7-1 which also demonstrates that the total resistance at any deflection is equal to the sum of the subelement forces. This subelement model can simulate the nonlinear-inelastic and hysteretic response of soils under lateral loading. Gapping (separation of soil from pile) and degradation (loss of resistance) are also available in the soil model.

In the analysis of test data to be presented later, the BMCOL 76 program (Matlock et al, 1981) could have been used. The BMCOL 76 program was used in pretest analysis but is less versatile when compared to SPASM because the soil support model is elastic, though nonlinear, and hysteretic unloading cannot be simulated. However, for monotonically increasing static loading, the results of the two programs are identical. The SPASM program was used to analyze all the test data because in some cases modelling of hysteretic soil unloading (sawtooth behavior) was required. For simpler cases where loading was monotonic and increasing in one direction only, the SPASM program was still preferred over BMCOL 76 because of the convenience associated with using only one consistent set of input format.

Input. The computer model and input are shown in Fig. 7-2. The soil profile shown in Fig. 3-9 was used. A friction angle of 35 degrees was assigned to the sandy material and the shear strength of the silty clay was taken as 1.5 psi (10.3 kPa). A constant submerged unit weight of 50 pcf (1.8 kN/m^3) and a strain at 50 percent of maximum stress of 2 percent were used. In deriving the p-y curves for this layered soil system, the equations recommended by Matlock (1970) were used for the clay. For sand, the equations provided by Reese et al (1974) were approximated by equivalent but simplified numerical p-y values. These numerical p-y curves were tabulated and reported by Bogard and Matlock (1970). For both soil types, the overburden pressure was accumulated to the depth at which the p-y curve was calculated. The resulting static p-y curves are shown in Fig. 7-3.

Pile loading in the computer solution was controlled by movable supports located at the same elevations as the two load struts. As noted in Section 4.3, deflection measurements were not taken directly at the load strut levels but were monitored on an auxiliary bar at elevations of 39 inches (99 cm) and 87 inches (221 cm) above the ground surface. Deflections at the load strut levels, which were subsequently used in the SPASM analyses, were calculated from measurements taken at the above two elevations. Both LVDT and dial gage readings were used. In some cases, the analog plots of load-deflection for the lower strut location of Pile 2 were also used. As discussed later, the deflections measured by Piles 1 and 2 were quite similar, therefore only the deflections recorded by Pile 2 were used in SPASM.

7.3 Detailed Analytical Procedure

A series of SPASM solutions were obtained using the input data described in Section 7.2. The SPASM solutions were then used to compare with the field measurements to assess the validity of the p-y curves selected for soil support representation. The comparison was limited to pile-head load-deflection and selected sets of bending moment distribution. Presentation of bending moment comparisons in this section are concentrated on the data recorded at high load levels where large deflections existed near the ground surface resulting in nonlinear soil response. The high load levels are also important from a design point of view.

In these SPASM solutions, static p-y curves were used for the initial PRH test and cyclic p-y curves were used for the remaining tests. Later a decision was made to compare the pile-head load-deflection relationship for the same boundary condition using the static and cyclic p-y curves. So, the static p-y curves were also used in the FH test for comparison purposes.

The choice of static or cyclic p-y curves for the computer analysis was compatible to the concept used in the derivation of the original p-y criteria. The static nonlinear-elastic p-y curves represent the soil resistance versus pile deflection under the first monotonically increasing loading (virgin condition only). The cyclic p-y curves represent a lower bound of expected resistance under a continuously increasing level of cyclic loading. Cyclic curves are generated to deal with cyclic effects in design on a quasi-static equivalent of the full history of loading.

A total of four computer runs were made. Two runs were used to simulate the two PRH tests including the sawtooth behavior. The remaining two sets of solutions were used to simulate the equilibrated condition after soil degradation of each of the FH and FRH cyclic tests. A cycle-by-cycle evaluation was not attempted for the cyclic tests. The purpose of this analysis was to investigate the applicability of the API p-y criteria by comparing conventionally calculated versus measured pile-head behavior and bending moment distribution. A detailed consideration of continuous cycling of the test piles requires much time and is not normally considered in design.

In this set of SPASM solutions, some thoughts were given on how to model the pile-soil system from one test to the next. From previous load test experiences in cohesionless soils, when the pile is pushed one-way, soil moves down at the back side of the pile and is then compacted upon unloading. This form of soil movement is known as grain migration (Section 6.2). The SPASM model was necessary for the sawtooth loading, however, effect of grain migration is not duplicated directly by the soil support representation.

At the end of the initial static PRH test, a permanent set of about 0.3 inch (0.8 cm) was measured at the lower strut. This permanent set remained more or less stable throughout subsequent testing and was thought to be the result of grain migration and soil compaction upon initial loading and unloading. In order to include this permanent set, an expedient approximation was used by first starting each SPASM solution at zero load and moment; however, in presenting the results, the permanent set of 0.3 inch (0.8 cm) was added back to the deflection values. In some cases, residual pile-head loads were also added back to the load values for the load-deflection plots. A partial justification for this approximate procedure was that residual moments, deflections and loads were small.

A profile of residual bending moment measured immediately before the FH tests is presented in Fig. 7-4. This profile was recorded at a no-load condition. As shown in Fig. 7-4, the maximum bending moment recorded at the last strain gage station was about 600 kip-in (68 kN-m) which was about 7 percent of the calculated yield moment assuming steel yielded at 36 ksi (248,000 kPa). For the more important near surface strain gage stations, the recorded moments were much lower (about 1.5 to 3.0 percent) when compared to yield.

A more complicated alternative to include residual effects may have been to shift the individual p-y curves at the end of each loading series until full reaction and deflection compatibility is achieved. This procedure is complex and was not believed to be justified in view of the small residual loads, deflections and moments.

7.4 Evaluation of API p-y Criteria

After the input data and analytical procedure were selected, the SPASM program was used to perform the necessary computations. The following sections summarize the solutions of the SPASM computation and present the results of the comparison.

Partially Restrained-Head (PRH) Test. The measured and calculated lower strut load-deflection relationships are shown in Figs. 7-5 and 7-6 for the first and final PRH tests. Considering that there was no trial-and-error process involved in selecting the soil parameters for the derivation of the p-y curves, the match is surprisingly close. However, a discrepancy is observed between the measured and calculated curves during pile unloading by reverse deflection for the first PRH test. This difference might be attributed to (1) the lack of recorded deflection data to define more accurately the unloading path of the tests in the computer input, and (2) inadequacy of the SPASM soil model to simulate grain migration.

Three sets of bending moment distribution are presented in Figs. 7-7 through 7-9. The pile head condition at which these measurements were recorded are marked in the load-deflection plots (Figs. 7-5 and 7-6) as points A, B and C.

In general, the maximum bending moment dictates the design of the wall thickness schedule for a foundation pile. From the results of the SPASM solution and field observation shown in Figs. 7-7 through 7-9, the calculated maximum bending moments are conservatively slightly larger than the measured values. A comparison of the bending moment distributions showed a fairly close agreement between measured and calculated values. The results also indicate that the agreement improved as the applied load increased so that very good agreement exists at a maximum pile stress of about 13 ksi (90,000 kPa) (See Fig. 7-9).

Free-Head (FH) Test. Five FH tests were performed as the second part of the test program. These tests were all slow cyclic tests (about 1 to 8 minutes per cycle). The lower strut load-deflection curves for the first cycle of each test are plotted in Fig. 7-10. The reduction in peak lower strut load due to cyclic loading is also shown in Fig. 7-10.

Two computer simulations were performed using the static and cyclic p-y criteria. The cyclic p-y curves were used to approximate the equilibrated condition after cyclic loading of each of the five FH tests. The static p-y curves were used solely for comparison with the cyclic criteria. The increasing deflections recorded during the first cycle of the first series of FH tests and the peak deflections for the second through fifth series of FH tests were used in the computer input to simulate a continuously increasing one-way loading.

For clarification purposes, the SPASM solutions assumed zero load and moment before initial pile loading. In the later presentation of load-deflection plots, the deflection coordinates obtained from SPASM were shifted to match the permanent set resulted in the initial PRH test.

The two computed static and cyclic pile-head responses are shown as smooth curves in Fig. 7-10. The measured load-deflection characteristic after cyclic degradation is also included in Fig. 7-10. As shown in Fig. 7-10, a very good comparison existed for the degraded condition which is a valid comparison with the concept of the cyclic p-y criteria. The computed static and cyclic load-deflection curves based on API criteria are almost identical. However, the field measurements indicated a significant difference in pile-head response between initial loading and final equilibration after cycling for each of the five FH tests levels. Thus the API cyclic procedure is fairly well confirmed, but an anomaly exists with respect to static behavior.

Two sets of bending moment comparisons are presented for the FH tests. These comparisons are shown in Figs. 7-11 and 7-12. The measured data were taken from the third and last FH tests and are identified by points A and B in Fig. 7-10. The maximum bending stresses at these points, computed from the measured moments, were about 18 and 24 ksi (124,000 and 165,000 kPa). Only the measurements recorded by Pile 2 are presented. The data measured by Pile 1 were almost identical to Pile 2 and were therefore omitted in these plots.

As shown in Figs. 7-11 and 7-12, field measurements indicate that cyclic loading with controlled displacement tends to relax the bending moment. During controlled-displacement cyclic loading, the soil reactions along the pile length are gradually redistributed to account for the progressive loss of lateral resistance of the near-surface soils.

The comparison of bending moment distributions is limited to the equilibrated (end of cycling) cases. From the results shown in Figs. 7-11 and 7-12, the calculated maximum positive moments are conservatively larger than the measured values. Still, the agreement is quite good between the measured and calculated distributions.

Fully Restrained-Head (FRH) Test. Three cyclic tests were performed under this boundary restraint. No relative lateral pile movement was allowed at the top load strut level for this boundary condition. The comparison of the lower strut load-deflection relationship is shown in Fig. 7-13. For the same reasons outlined in Section 7.3, no attempt was made to trace the complete loading history in the SPASM solution. A lower-bound load-deflection curve was calculated using the measured increasing deflections of the first cycle of the first test and peak deflections of the remaining two tests as input.

Similar to the FH tests, zero load and moment were assumed to exist before pile loading in the SPASM input. The calculated load-deflection curve was then shifted to account for the permanent set of 0.3 inch (0.8 cm) which occurred after the initial PRH test and for the residual load of about 5.5 kips (24.5 kN) which was recorded at the beginning of this test.

The measured and calculated load-deflection relationships at the lower strut location are shown in Fig. 7-13. From the measurements, it appears that the soil had already equilibrated under the continuous cyclic motion of the preceeding FH tests. The lack of significant changes during cycling is also due to the domination of the fully restrained boundary condition; this case is far less sensitive to soil resistance variations than the FH condition. The prediction of load-deflection relationship was also quite good as shown in Fig. 7-13.

Two sets of bending moment plots are shown in Figs. 7-14 and 7-15. These data were obtained from the last two FRH tests. The calculated bending moment distribution produced a very good fit for the measured curves.

The FRH boundary condition is actually an unusual and extreme case and is not typical of conventional offshore structure foundations. Nevertheless, it is instructive and therefore was included in the test program. If the pile head comparisons between predicted and measured are good for the two extreme cases of FH and FRH boundary conditions, the p-y procedure should be valid for immediate cases.

7.5 Soil Resistance Variation During Cyclic Loading

Discussion of this topic is based on the data collected during the displacement-controlled FH tests. For a typical pile-supported jacket-type structure, most of the nonlinear soil response and variation under cyclic loading occurs in the upper 3 diameters of depth below the ground surface. A discrete evaluation of the soil response at the near-surface elevation was impossible and was not intended for this study. However, a study of the overall cyclic effect was achieved by monitoring the pile loading characteristic above the ground surface. A plot of lower strut peak load versus time for all the FH tests is presented in Fig. 7-16.

As mentioned in Section 4.4, the cyclic tests were controlled manually or by the HP microcomputer. In the manual mode, a complete data set, including deflections, loads, bending moments and pore pressure, were collected at the operator's command. In the computer-controlled mode, deflection and load data were collected at each peak major and minor deflections until the specified number of cycles was reached;

when the specified number of cycles was reached, a complete set of data was recorded. Both the manual and computer-controlled modes were applied in the FH tests. The time of testing at which the manual control was exercised is shown in Fig. 7-16.

As shown in Fig. 7-16, the soil resistance increases between cycles when the pile is loaded manually. In the manual mode, the load was applied incrementally by controlling the pressure of the hydraulic rams. Frequent complete data set were taken during loading in the manual mode. This manual operation to the peak deflection consumed a total of almost 50 minutes for the first FH test and 12 minutes for the third and final FH tests. The long test duration for the first FH test was a result of frequent interruptions to examine the data and to check out the loading system and data acquisition. When pile loading was controlled by the microcomputer, the loading time varied from about 1 to 8 minutes per cycle. At this loading rate, the measured data indicated a gradual reduction of soil resistance to a minimum value. This reverse rate effect of increase in soil resistance at slow loading rate is not very significant and may be related to small variation of pore pressure of the near-surface soils.

A different sort of decay of soil resistance was observed during the second FH test. The peak strut load measured at the first cycle was higher than expected but it was followed by a rapid decay at the second and third cycles. According to Fig. 7-16, degradation of soil resistance was not completed for the first FH test. The reserve resistance from the first FH test led to the high applied load measured at the second FH test which was performed at larger deflection limits.

The ratio of the degraded to peak lower strut load ranged from 0.72 to 0.93 for the FH tests. The minimum ratio was measured during the second FH test and the maximum was measured during the fourth test. The average value is 0.83 for all five tests. This observed ratio is significantly different from the pile-head responses calculated using the static and cyclic p-y criteria (See Fig. 7-10).

In the Seal Beach tests, there were other factors that may have contributed to soil resistance degradation. One obvious symptom was the subsidence of surficial soils around the piles due to grain migration which was observed in the field and reported in Section 5.5. Other factors may have included scour around the test piles, and pore water pressure build-up of the surficial soils at higher loading rates. The last two factors are speculative and are not supported by field evidence.

7.6 Comparison of Pile 1 and Pile 2 Behavior

There were primarily two reasons to compare the behavior of the two test piles. The first reason was to examine the effect of the denser soil around Pile 1 caused by previous vibration and shown by the CPT profiles (see Fig. 3-4). The second was to justify the validity of the measurements by comparisons between the two piles.

Both of the above objectives were accomplished by comparing the deflections measured by the four LVDT's located at two different elevations of each pile and the bending moment distributions. The measured deflections for the initial PRH and FH tests are shown in Figs. 7-17 and 7-18, respectively. Measurements from the FRH and the final PRH tests were not included because the conclusions drawn from the reported data reflected similar behavior.

From the initial static PRH data (Fig. 7-17), Pile 1 recorded a slightly smaller deflection when compared to Pile 2. This probably was a direct influence of the denser soil near the ground surface adjacent to Pile 1. In the FH test, only the peak major deflections at the first and final cycles of each of the five cyclic tests were plotted in Fig. 7-18. The deflection monitored during these two tests are, for all practical purposes, quite similar.

As shown in Fig. 7-18, a smaller deflection was recorded for Pile 1 at the beginning of the FH cyclic tests. Similar to the initial static PRH test, this smaller deflection was caused by the denser soil around Pile 1. However, as cyclic soil degradation occurred progressively in each of the FH tests, Pile 1 no longer deflected less than Pile 2. This indicated that cyclic degradation may have loosened the original dense structure of the soil around the top of Pile 1.

Several comparisons of bending moment distributions were illustrated in Section 6.3 and earlier in this section. Again, the agreement was generally good for the two piles. Bending moment is less sensitive to local variation of soil pressures such as the initially higher soil resistance against Pile 1. The good agreement between the two piles was expected because the bending moment distribution is dominated by the type of boundary constraints. At all times during testing, the two test piles were subjected to equal and opposite loading. With equal resulting deflections, the boundary restraints would be equal.

7.7 Boundary Conditions

Three different boundary restraints which covered a wide range of possible conditions were used in this study. The pile-head responses and bending moment distributions were totally different for these three cases. As expected, the stiffness from the load-deflection relationship measured at the lower load strut elevation increased with additional pile-head restraint. The shape of the bending moment distribution was similar for the PRH and FRH tests. Maximum bending moment generally occurred at the point of load application. For the FH tests, the bending moment is zero at the lower load strut level and the maximum value occurred at about 5 times the pile diameter below the ground surface. These results demonstrate the importance of proper assignment of boundary condition in lateral pile analysis.

There are a few cases where foundation piles are completely free (unrestrained) above the mudline. Two examples are anchor piles or single pile moorings. For most offshore structures, some form of restraint always exists above the mudline and should be properly modelled in any analysis. This includes the special case in which the foundation piles are driven and left loose inside the jacket legs. The method for soil characterization in a pile solution may be important, but the physical modelling of the boundary condition, which dominates the bending moment response, is of primary significance for a successful design.

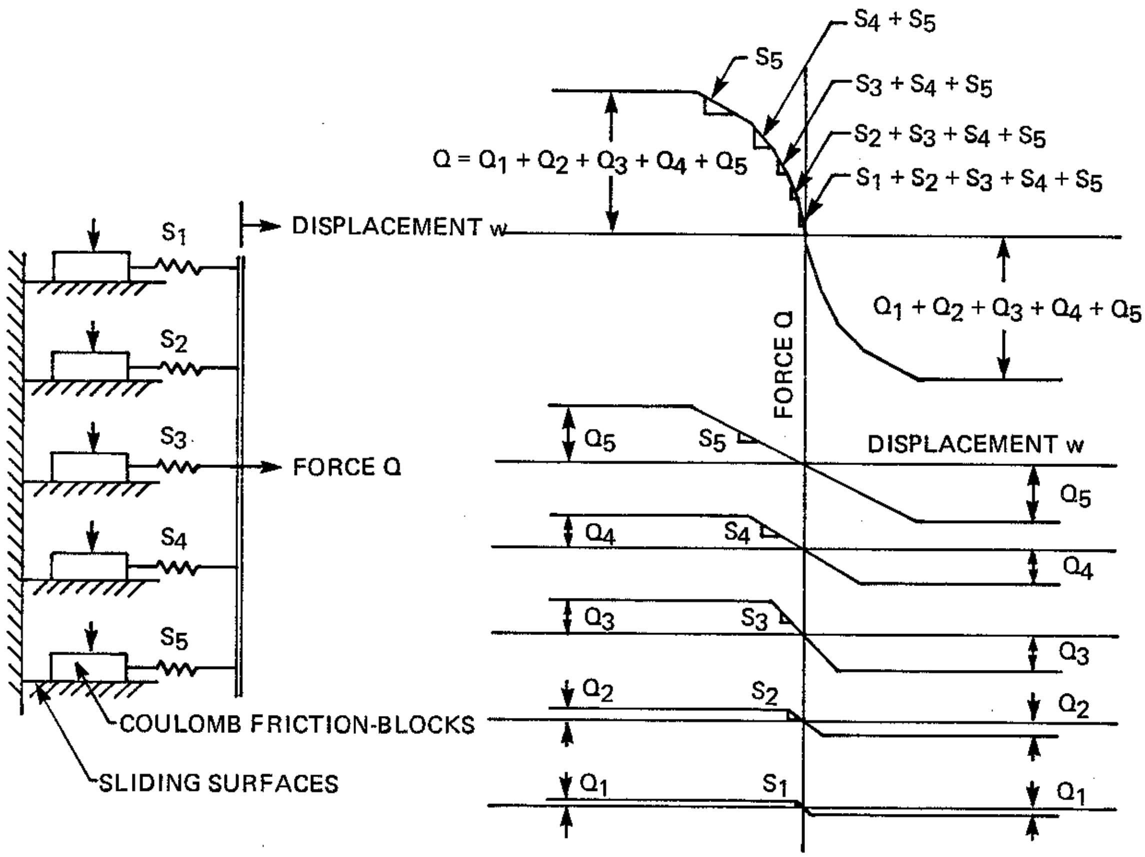
7.8 Discussion

Further input for lateral pile design in sand was made by Barton et al (1983). Barton's results, which were obtained in a centrifuge, confirmed that the API cyclic p-y curves are applicable for analyzing offshore pile foundations under cyclic lateral loading for sandy materials.

In this study, the p-y curves for the layered soil were derived using both the soft clay and sand criteria. The equations in each criteria were used for the corresponding soil type. In these equations, the effective overburden was calculated by the accumulation of vertical pressures to the depth at which the p-y curve was located. Since the field measurements and analytical solutions were observed to be dominated by the sand, conclusions drawn from this study are more a justification of sand criteria than soft clay criteria.

The computer program SPASM (Matlock et al, 1979) was used in the prediction of lateral pile behavior in this study. From the results reported in this chapter, it appears that the SPASM program is a useful tool in analyzing and back-fitting data measured during this series of load tests. The flexibility in input and soil modeling allowed efficient processing of the field results and subsequent prediction of the behavior of the Seal Beach test piles.

In summary, the conventional p-y representation of the layered soil at the Seal Beach test site appeared to be quite adequate for predicting the lateral behavior of the test piles.

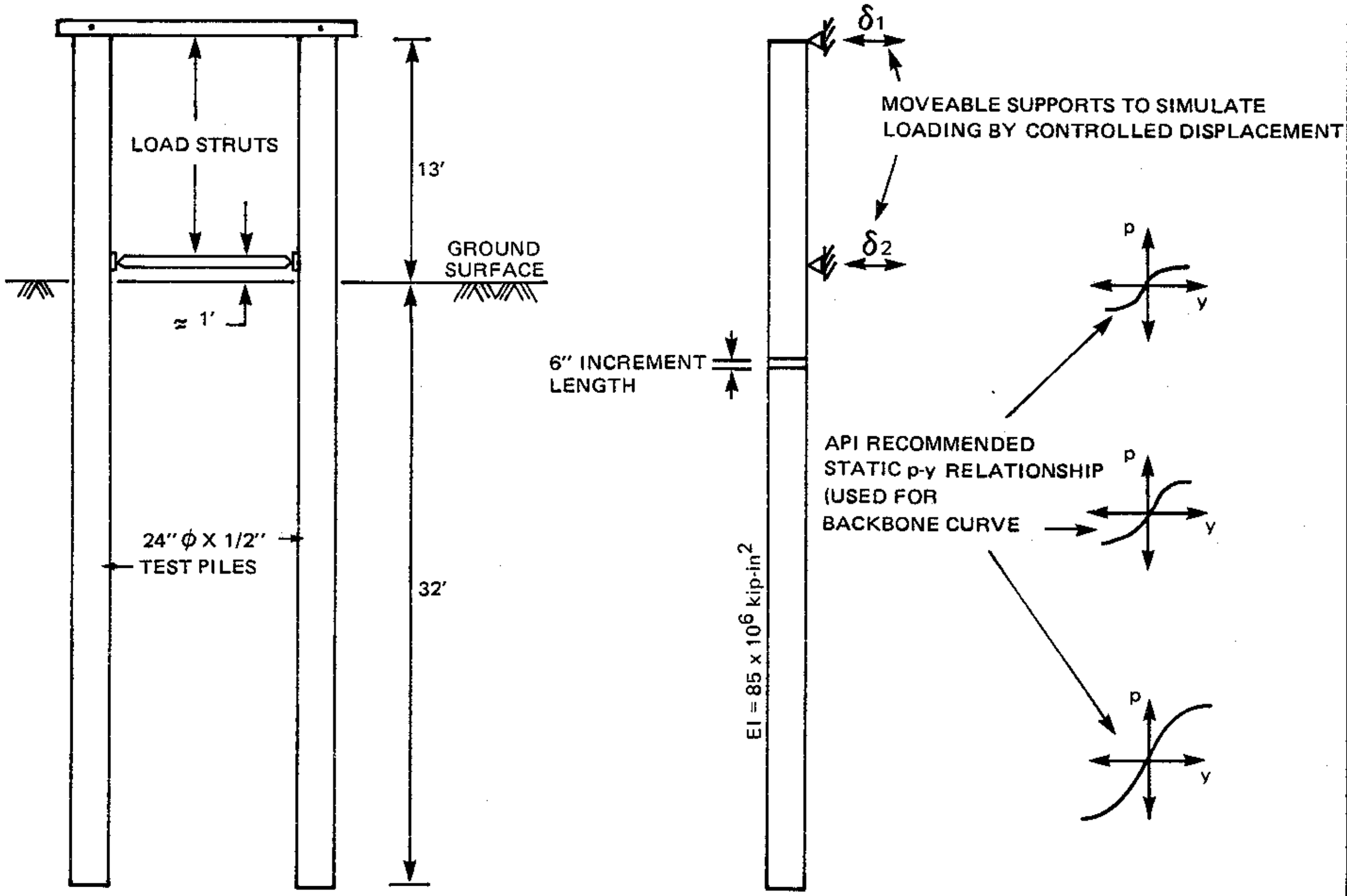


Ertec — The Earth Technology Corporation	PROJECT NO.: 82-205 SEAL BEACH

SUB-ELEMENT MODEL USED IN SPASM ANALYSIS (AFTER CHAN AND MATLOCK, REF. 6)

8-83

FIGURE 7-1



(A) TEST PILE ASSEMBLY

(B) COMPUTER MODEL

Ertec
The Earth Technology Corporation

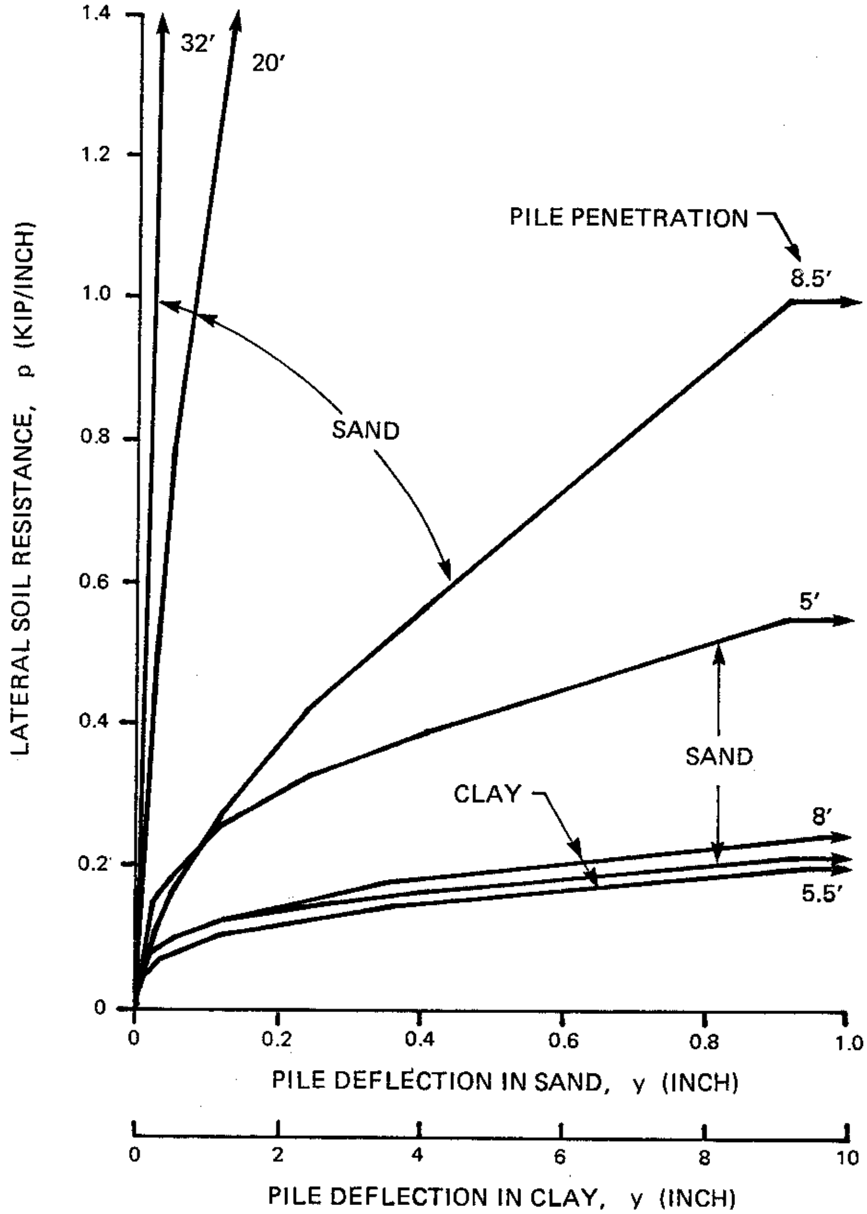
PROJECT NO.: 82-205

SEAL BEACH

PROBLEM DESCRIPTION FOR
SPASM ANALYSIS TO
SIMULATE PILE LOAD TEST

8-83

FIGURE 7-2



Ertec
The Earth Technology Corporation

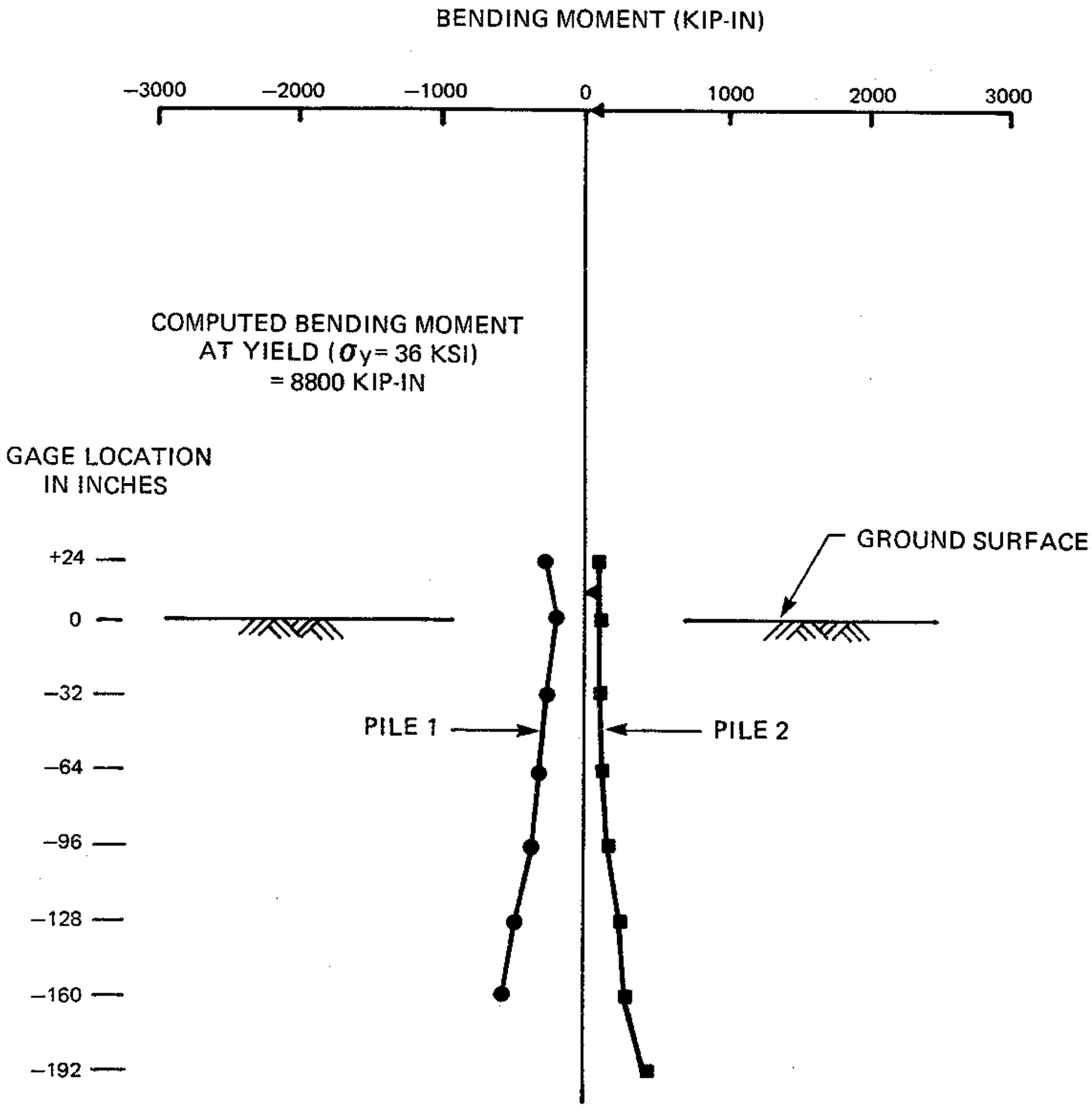
PROJECT NO.: 82-205

SEAL BEACH

STATIC SOIL RESISTANCE-PILE DEFLECTION RELATIONSHIPS USED IN SPASM ANALYSIS

8-83

FIGURE 7-3



Ertec
The Earth Technology Corporation

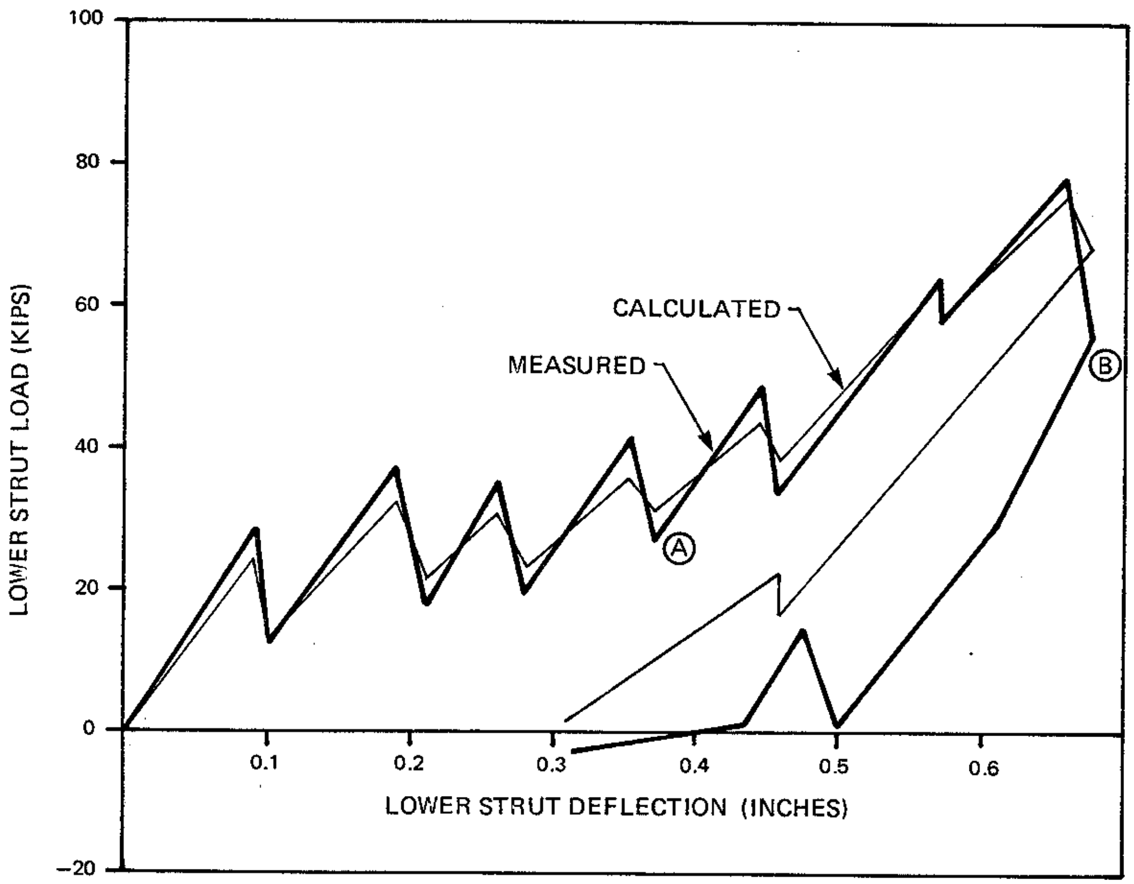
PROJECT NO.: 82-205

SEAL BEACH

RESIDUAL MOMENT MEASURED RIGHT BEFORE THE FREE HEAD TESTS

8-83

FIGURE 7-4



Ertec
The Earth Technology Corporation

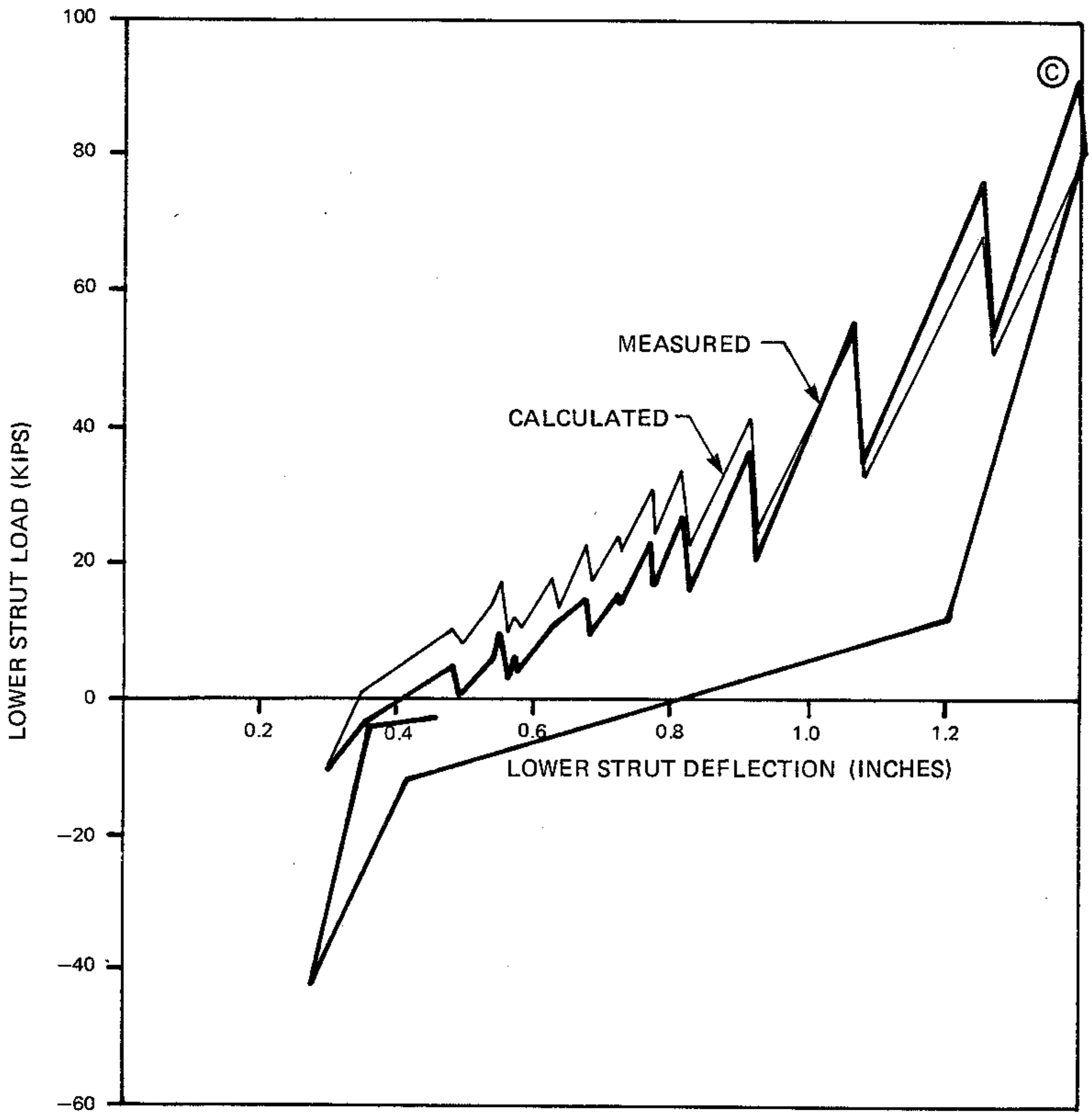
PROJECT NO.: 82-205

SEAL BEACH

CALCULATED VERSUS MEASURED LOAD-DEFLECTION RELATIONSHIPS OF INITIAL PARTIALLY RESTRAINED-HEAD TEST

8-83

FIGURE 7-5



Ertec
The Earth Technology Corporation

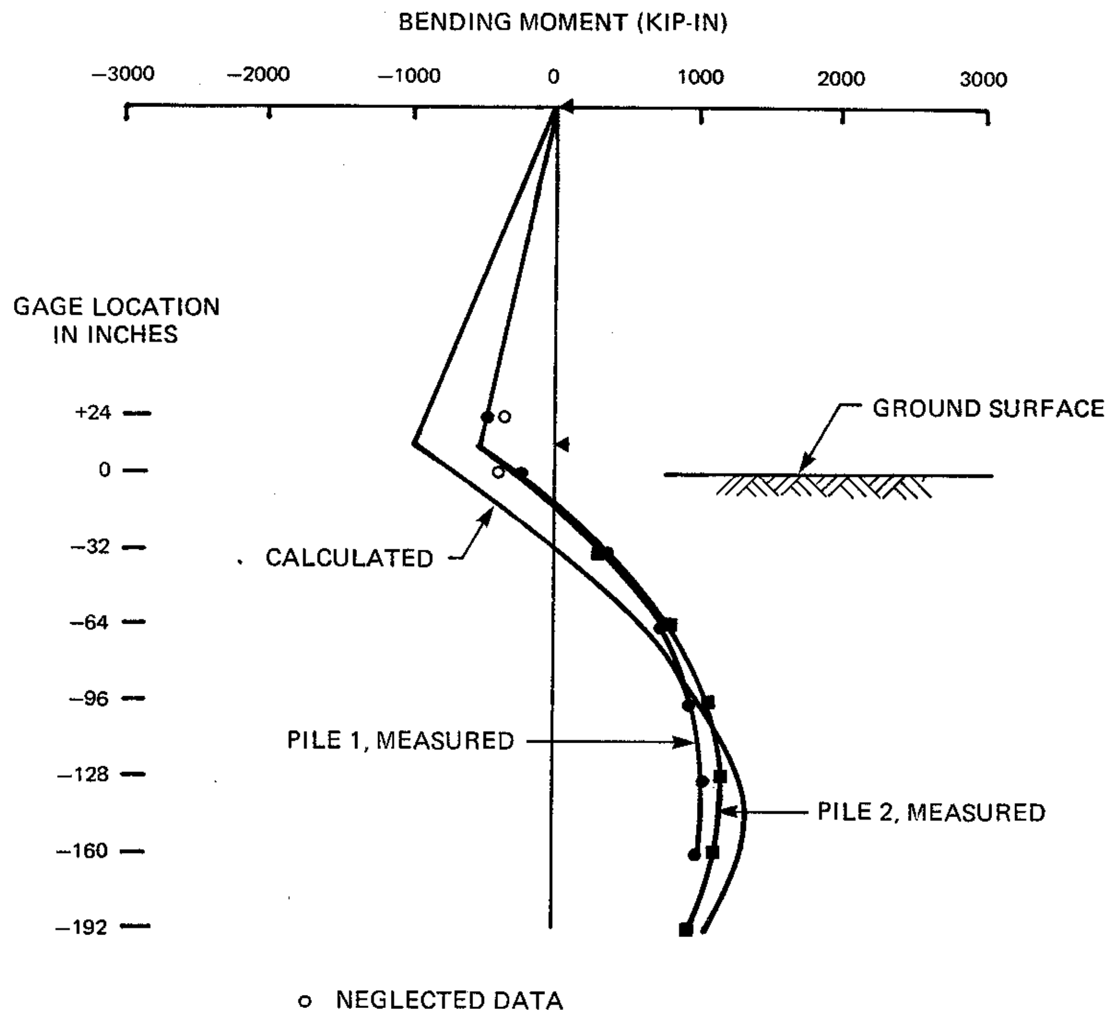
PROJECT NO.: 82-205

SEAL BEACH

CALCULATED VERSUS MEASURED LOAD-DEFLECTION RELATIONSHIPS OF FINAL PARTIALLY RESTRAINED-HEAD TEST

8-83

FIGURE 7-6



Ertec
The Earth Technology Corporation

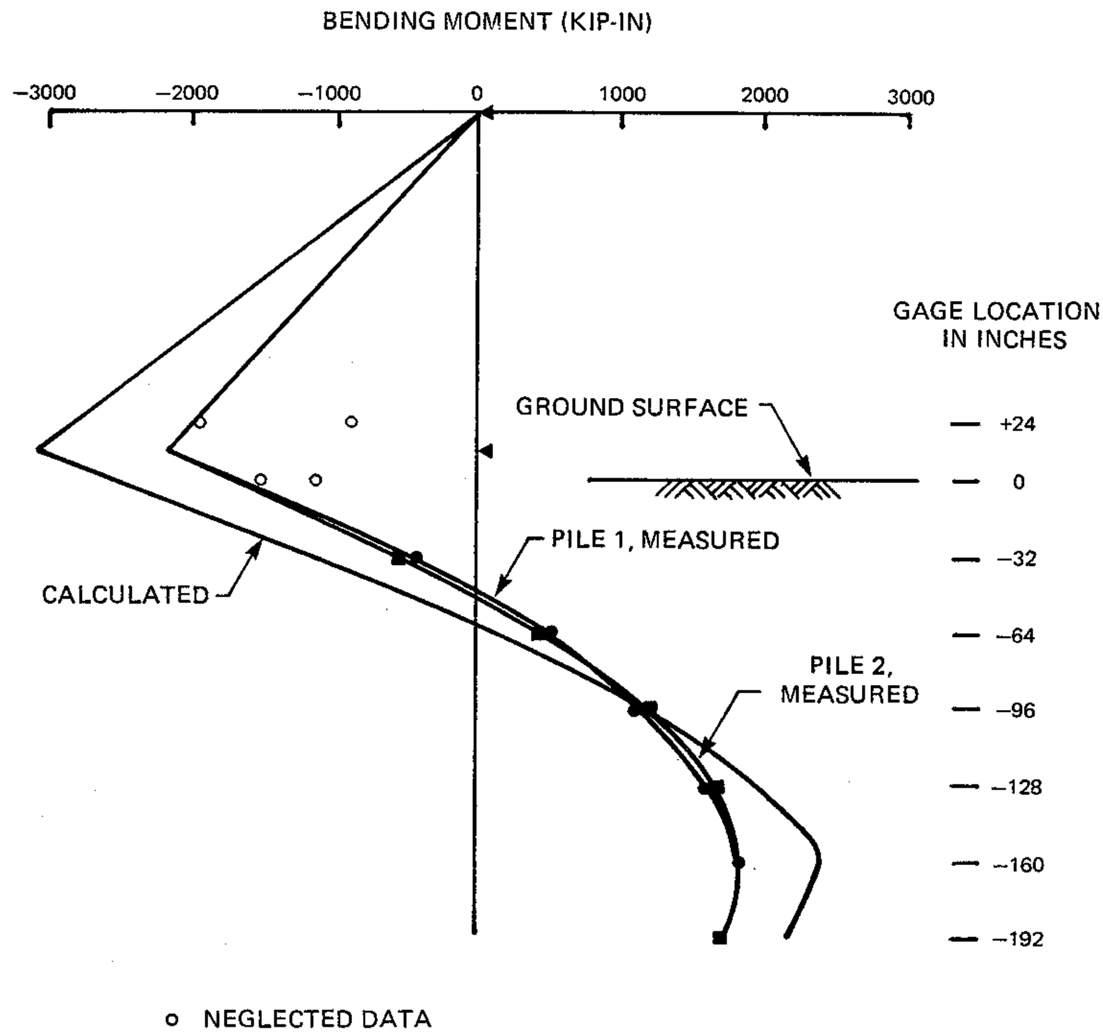
PROJECT NO.: 82-205

SEAL BEACH

CALCULATED VERSUS MEASURED BENDING MOMENT DISTRIBUTION AT POINT A OF INITIAL PARTIALLY RESTRAINED-HEAD TEST

8-83

FIGURE 7-7



Ertec
The Earth Technology Corporation

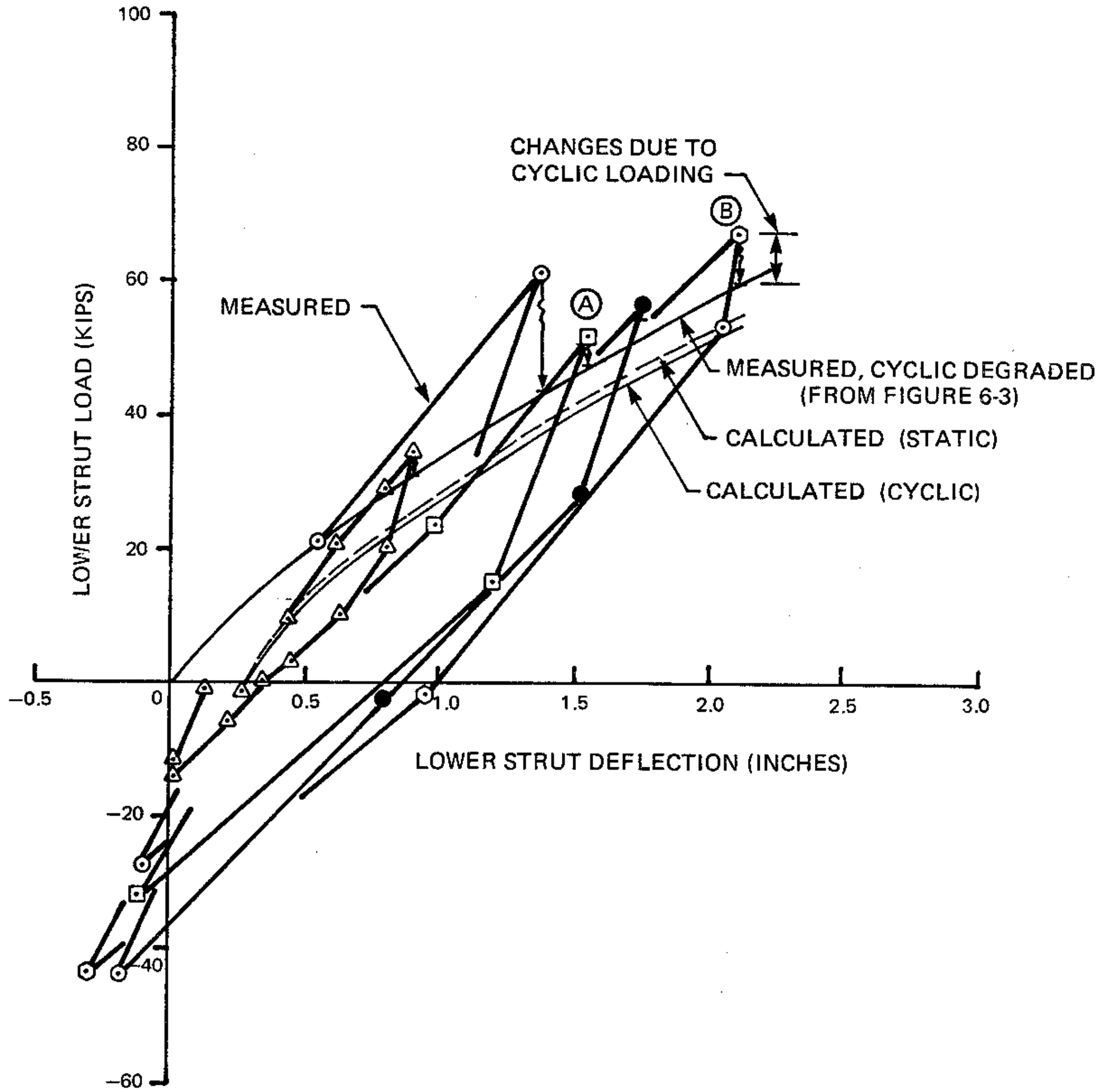
PROJECT NO.: 82-205

SEAL BEACH

CALCULATED VERSUS MEASURED BENDING MOMENT DISTRIBUTIONS AT POINT B OF INITIAL PARTIALLY RESTRAINED-HEAD TEST

8-83

FIGURE 7-8

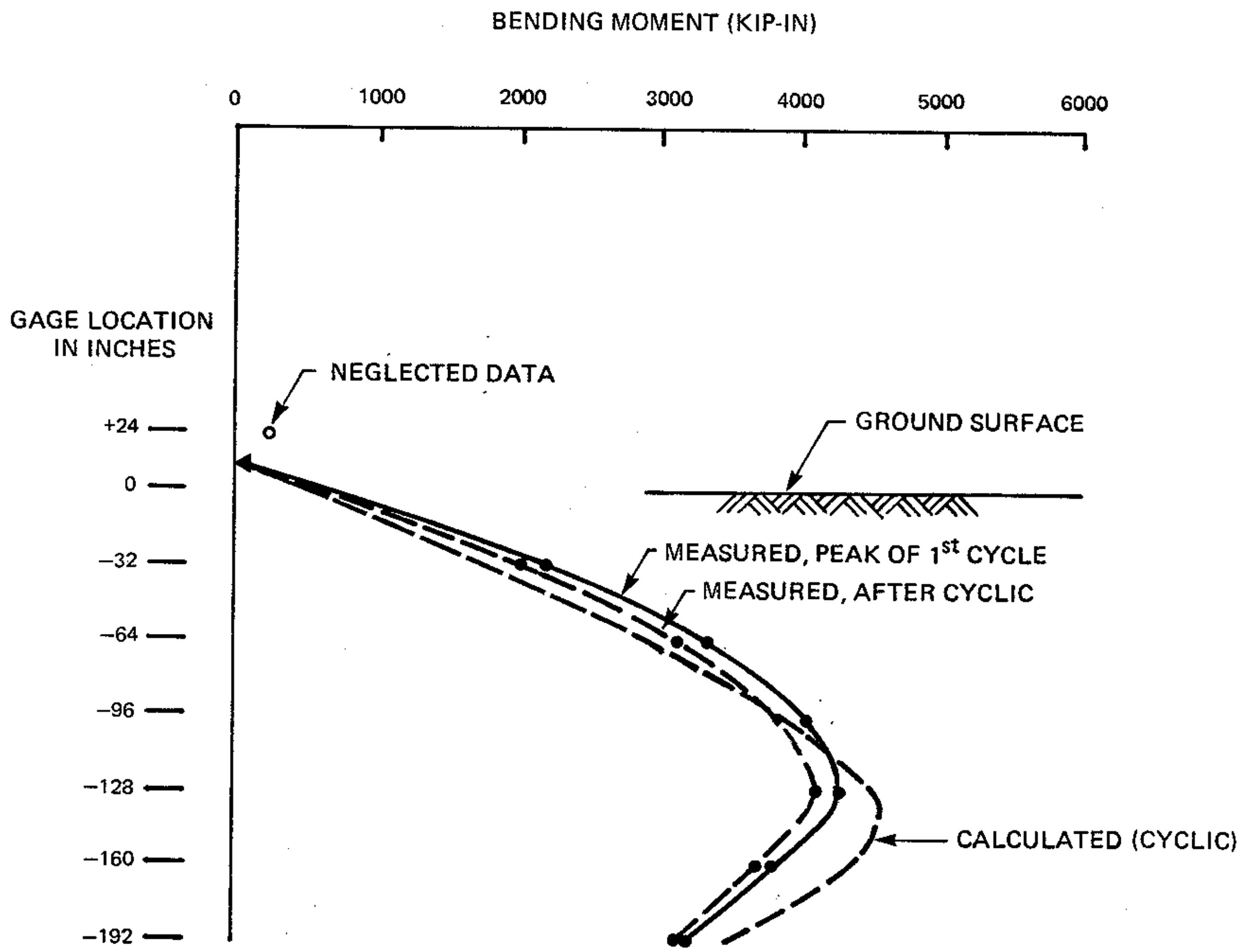


Ertec The Earth Technology Corporation	PROJECT NO.: 82-205 SEAL BEACH

CALCULATED VERSUS MEASURED LOAD-DEFLECTION RELATIONSHIPS OF FREE-HEAD TESTS

8-83

FIGURE 7-10



Ertec
The Earth Technology Corporation

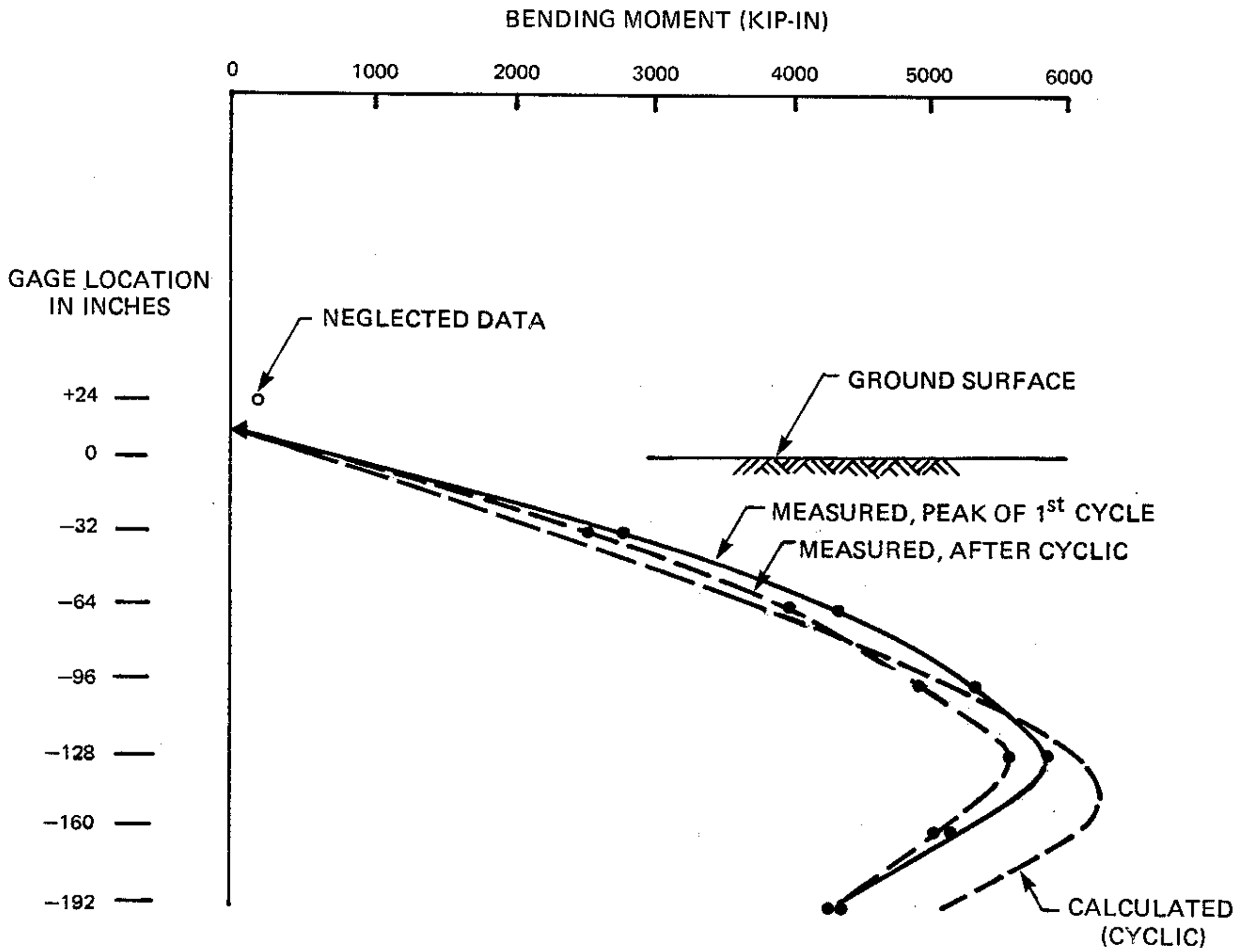
PROJECT NO.: 82-205

SEAL BEACH

CALCULATED VERSUS MEASURED BENDING MOMENT DISTRIBUTIONS FOR PILE 2 AT POINT A OF FREE-HEAD TEST

8-83

FIGURE 7-11



Ertec
The Earth Technology Corporation

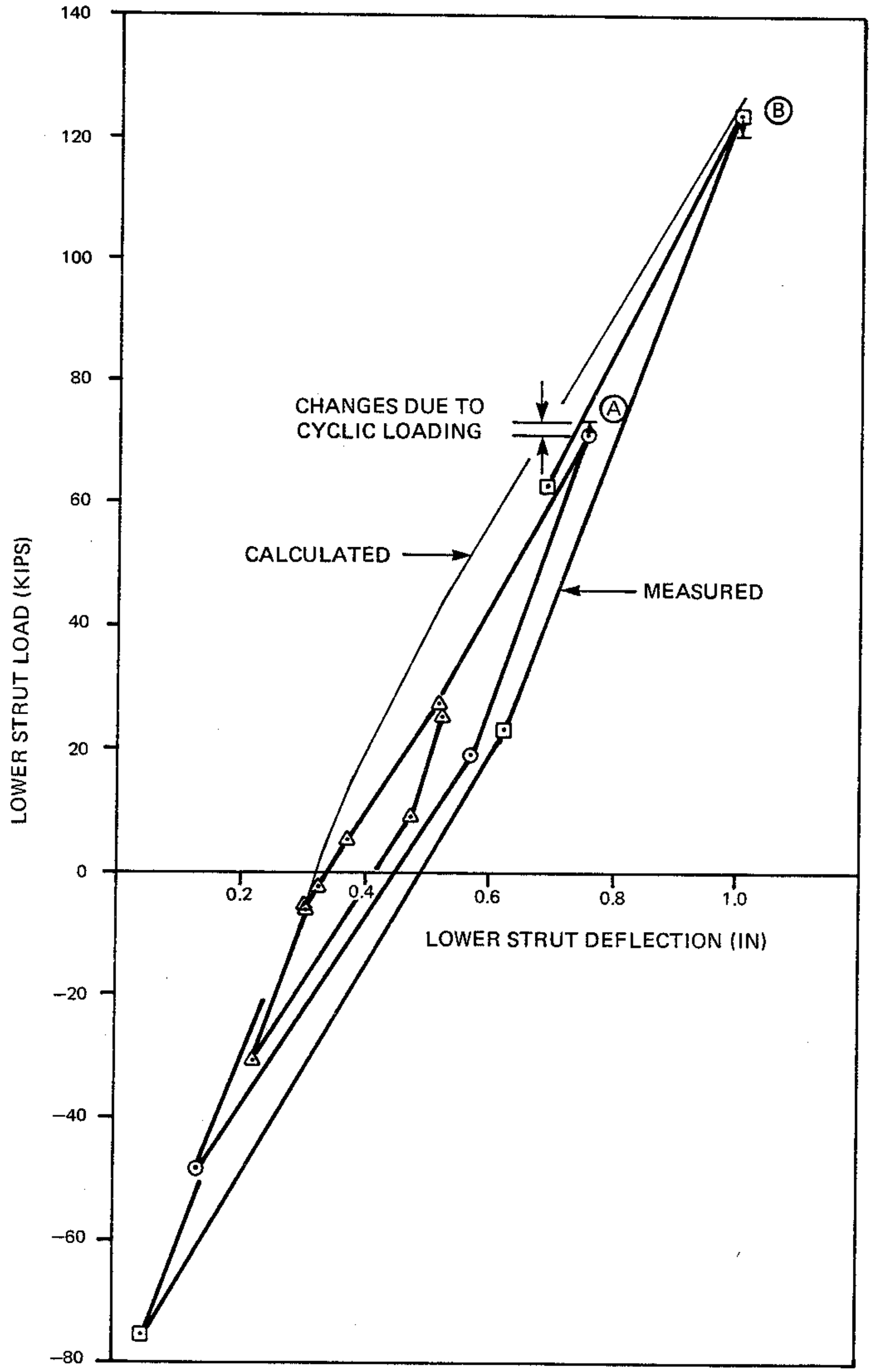
PROJECT NO.: 82-205

SEAL BEACH

CALCULATED VERSUS MEASURED BENDING MOMENT DISTRIBUTIONS FOR PILE 2 AT POINT B OF FREE-HEAD TEST

8-83

FIGURE 7-12



Ertec
The Earth Technology Corporation

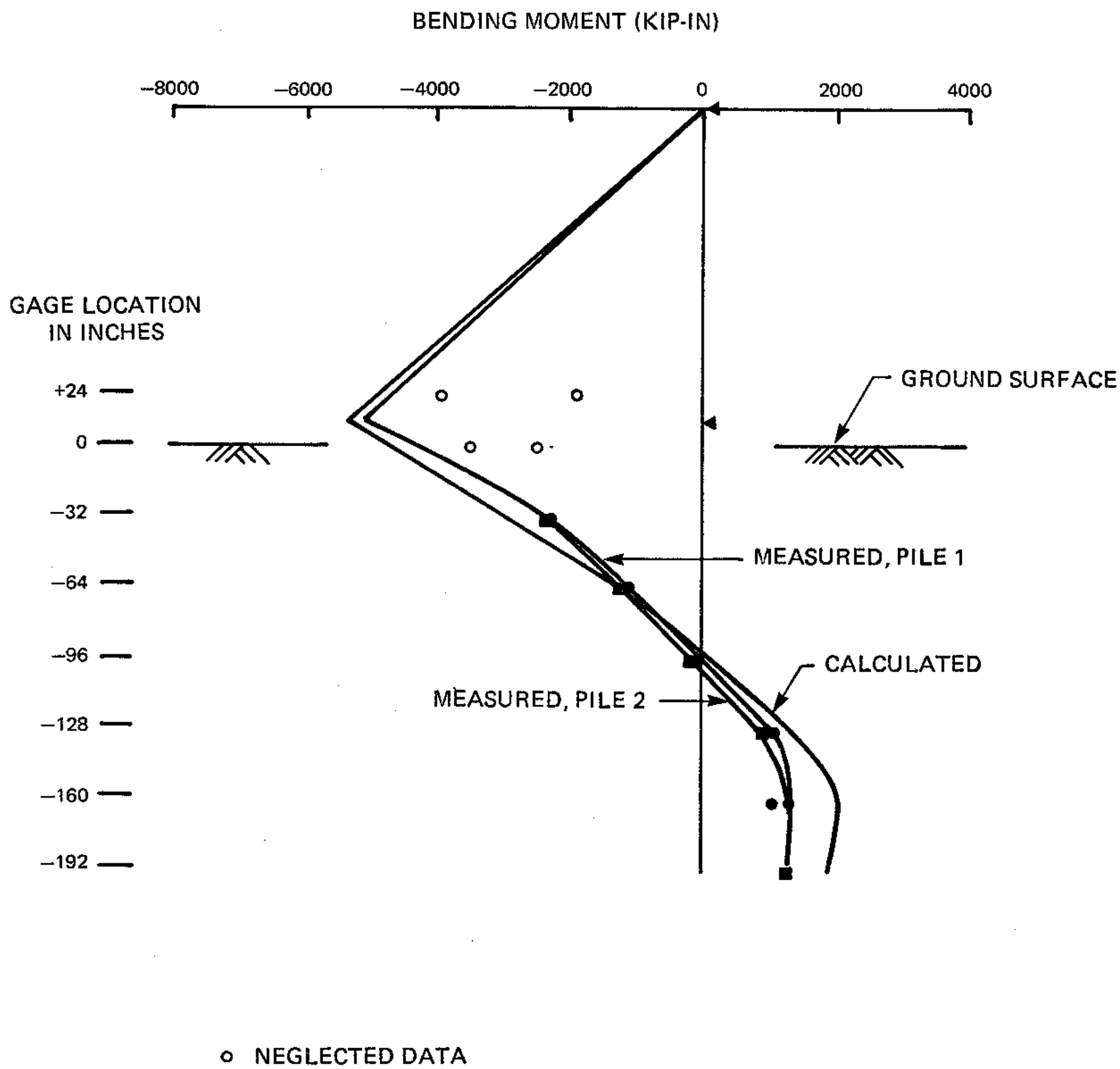
PROJECT NO.: 82-205

SEAL BEACH

CALCULATED VERSUS MEASURED
LOAD- DEFLECTION RELATIONSHIPS OF
FULLY RESTRAINED-HEAD TESTS

8-83

FIGURE 7-13



Ertec
The Earth Technology Corporation

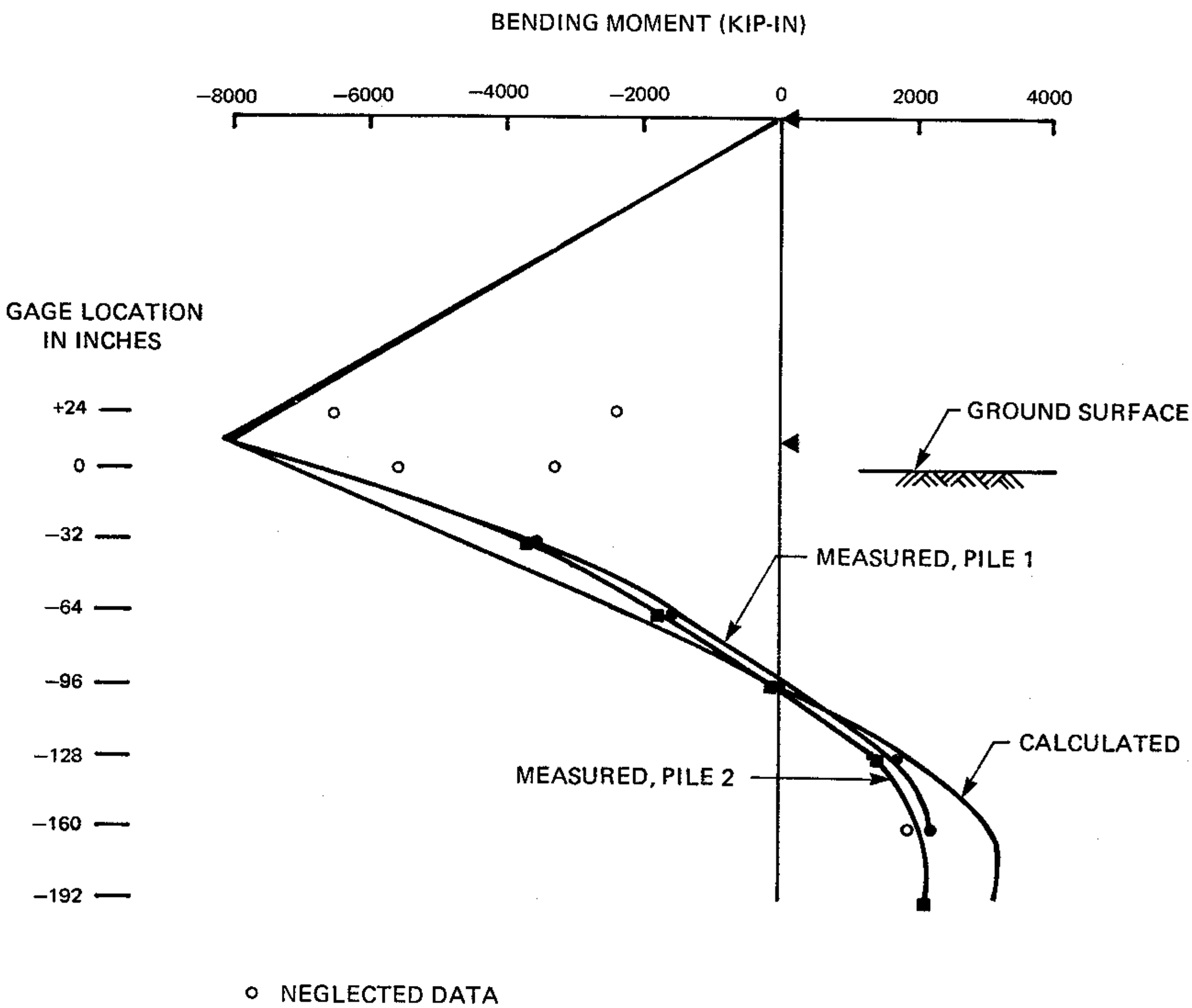
PROJECT NO.: 82-205

SEAL BEACH

CALCULATED VERSUS MEASURED BENDING MOMENT DISTRIBUTIONS AT POINT A OF FULLY RESTRAINED-HEAD TEST

8-83

FIGURE 7-14



Ertec
The Earth Technology Corporation

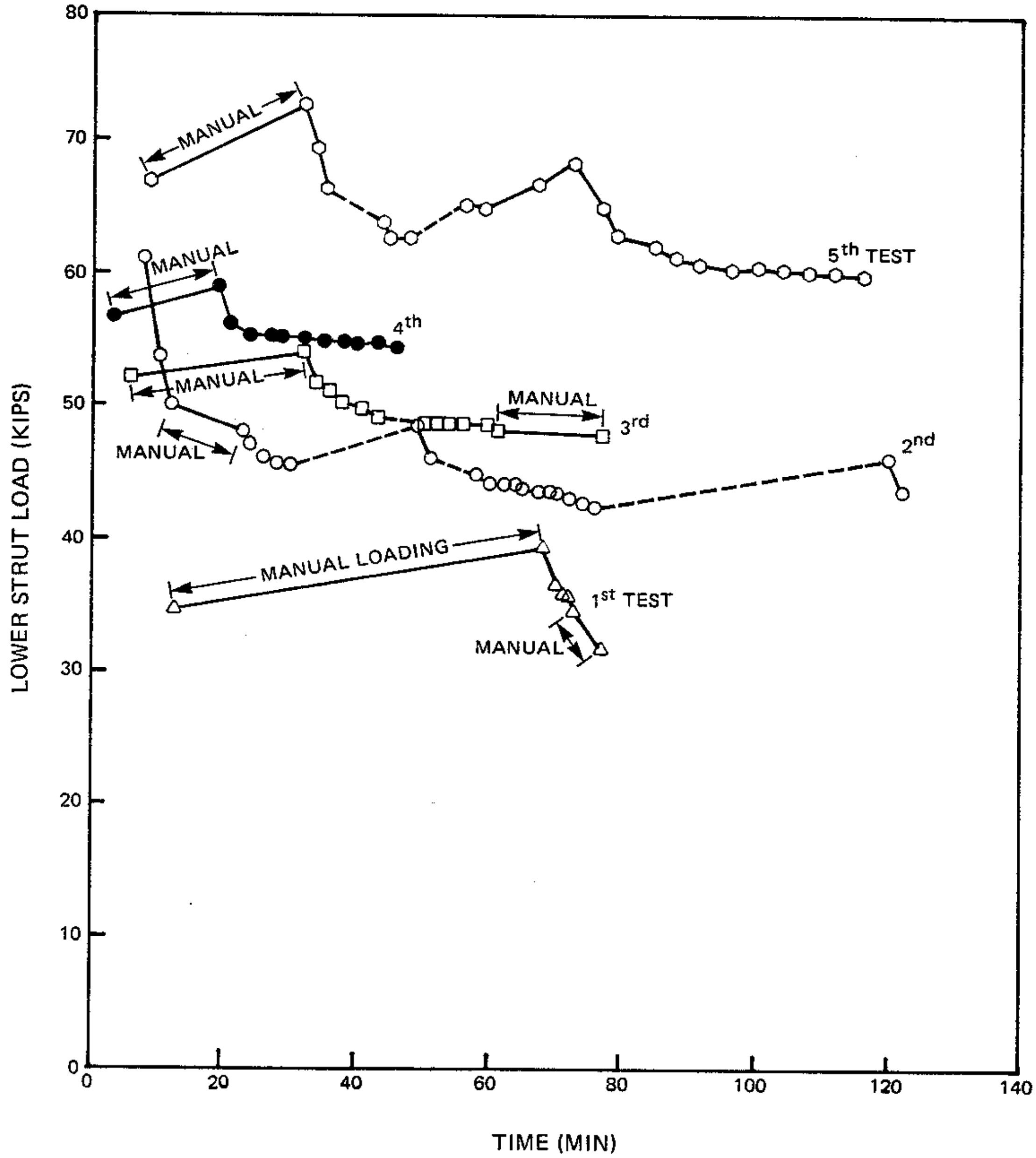
PROJECT NO.: 82-205

SEAL BEACH

CALCULATED VERSUS MEASURED BENDING MOMENT DISTRIBUTIONS AT POINT B OF FULLY RESTRAINED-HEAD TEST

8-83

FIGURE 7-15

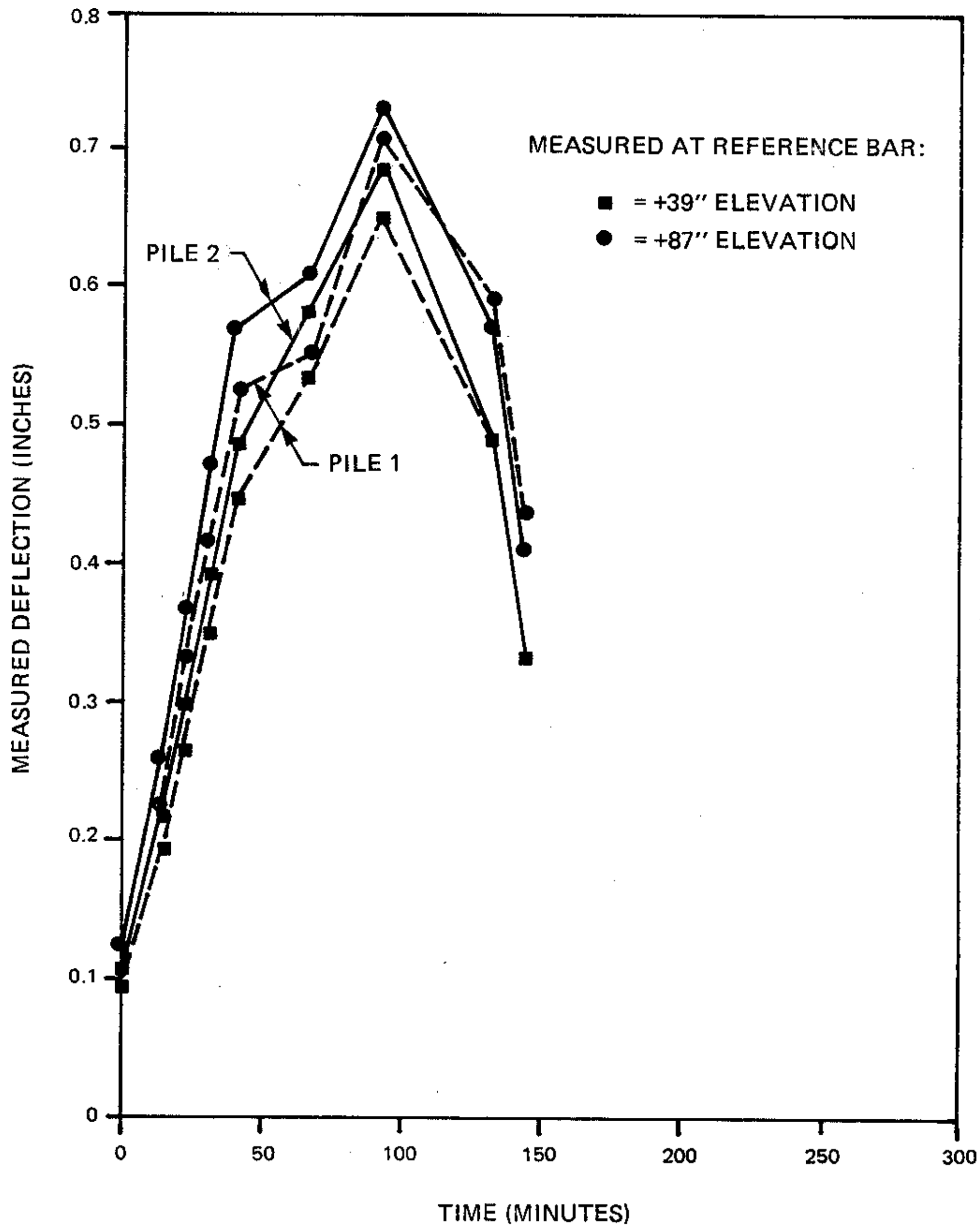


MANUAL LOADING – LOADING CONTROL BY MANUAL SWITCH ON PANEL BOX (NOT COMPUTER CONTROLLED)

Ertec The Earth Technology Corporation	PROJECT NO.: 82-205 SEAL BEACH

VARIATION OF PEAK LOWER STRUT LOAD DURING CYCLIC FREE-HEAD TESTS

8-83 FIGURE 7-16



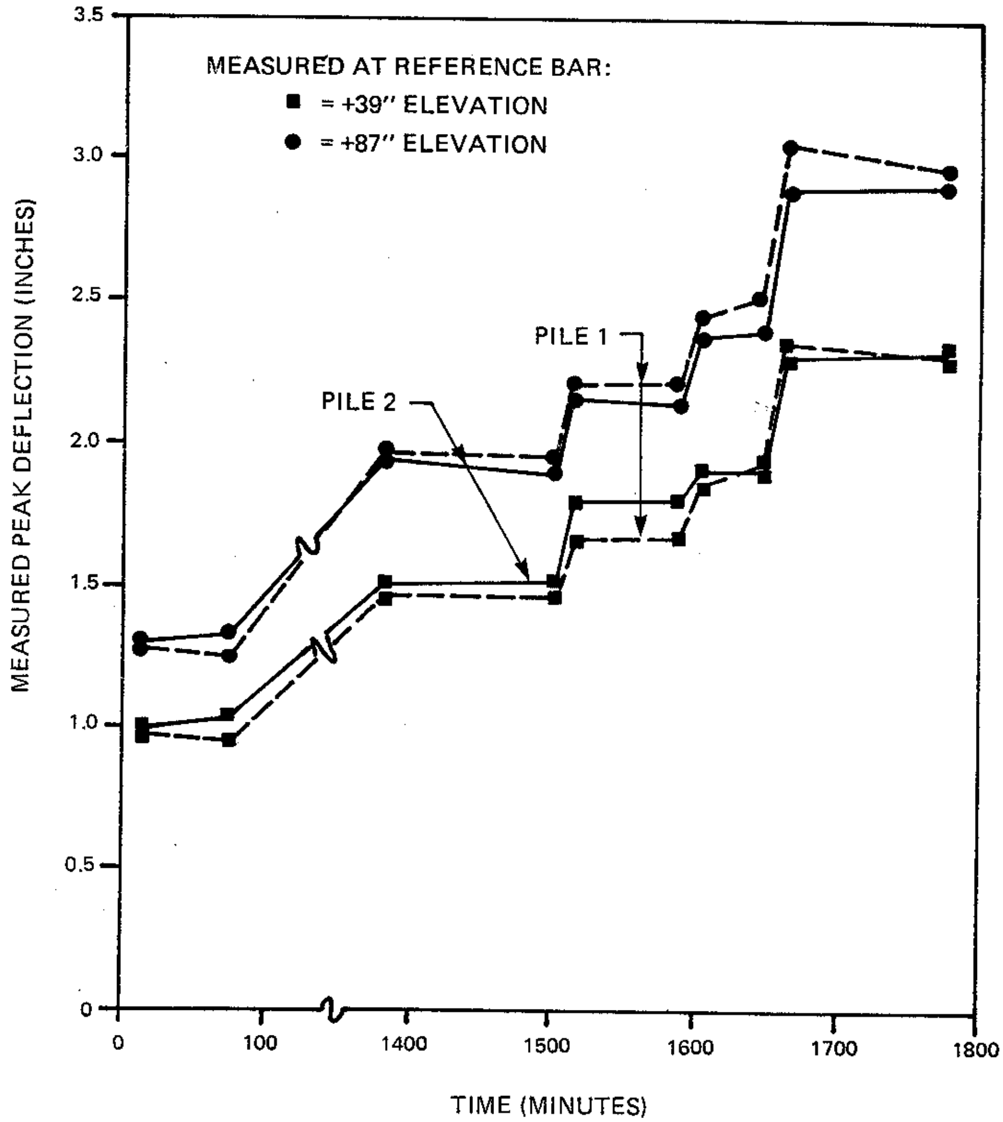
Ertec
The Earth Technology Corporation

PROJECT NO.: 82-205
SEAL BEACH

COMPARISON OF MEASURED DEFLECTIONS FOR PILES 1 AND 2 DURING INITIAL PARTIALLY RESTRAINED-HEAD TEST

8-83

FIGURE 7-17



Ertec
The Earth Technology Corporation

PROJECT NO.: 82-205

SEAL BEACH

COMPARISON OF MEASURED PEAK DEFLECTIONS FOR PILES 1 AND 2 DURING FREE-HEAD TESTS

8-83

FIGURE 7-18

8.0 SUMMARY AND RECOMMENDATIONS

8.1 Summary

Static and slow cyclic tests were performed on two instrumented piles. The piles were embedded in a predominantly silty sand deposit with a silty clay layer occurring from about 5 to 8 feet (1.5 to 2.4 m) below the ground surface. The piles were 24 inches (61 cm) in outside diameter with a wall thickness of 0.5 inch (1.3 cm). The piles were driven in place to a final depth of 32 feet (9.8 m) below the ground surface.

The piles were instrumented to measure bending moment along the shaft. Pile-head load and deflection were also recorded. No significant pore pressure variations were observed during loading. Ten tests were performed in a duration of three days. All tests were displacement-controlled. The piles were tested under three boundary conditions: (1) partially restrained-head (PRH), (2) free head (FH), and (3) fully restrained-head (FRH).

Measured pile-head load-deflection relationships and bending moment distributions were plotted for these tests. Data interpretation included discussion of cyclic degradation, effects of boundary conditions and soil strength variations. Results from the computer simulation of the field tests were also presented and were used to compare with field measurements.

The API recommendations do not include a procedure specifically for the formulation of p-y curves in layered soils. In this study, the p-y curves were derived using both the sand and clay criteria recommended by API. Each criterion was used independently for the corresponding soil type. Overburden pressure was accumulated to the depth of the p-y curve regardless of the soil types or criteria. From the data presented in Chapter 7 of this report, this procedure appears to work quite well for cyclic loading.

The following summarizes the principal findings of this study:

(1) The test set-up successfully duplicated the three desired boundary conditions and instruments generally functioned satisfactorily.

(2) There was no appreciable difference in the response of the two piles although the sand around the previously vibrated pile was shown by Cone Penetration Testing to be significantly densified in the upper few feet.

(3) After the initial test loading (PRH), the permanent deflection of the pile near the ground surface was about 0.3 inch (0.8 cm), due to sand falling in behind upon loading. Little further permanent change was noted throughout the remainder of the test program.

(4) Under FH testing, at each new load level some amount of progressive load reduction before final stabilization was observed with cycling. With the fully restrained-head (FRH) loading, very little progressive change was noted during cycling.

(5) The very significant influence of pile-head restraint was demonstrated, with results generally more influenced by boundary conditions than by soil variations.

(6) Very slight increases in soil resistance were noted for slower tests, in the range of 12 to 50 minutes per cycle.

(7) No significant elevations of pore pressure were observed during the testing.

(8) The behavior of the piles was primarily influenced by the sand layers. To a satisfactory degree, the soft clay was accounted for in the analysis by using conventional soft clay criteria at appropriate depths along the pile.

(9) The SPASM program, with a nonlinear and inelastic soil support model, proved to be a versatile tool for lateral pile analysis where nonlinear-elastic supports are inadequate.

(10) There is very little difference between API criteria for static versus cyclic loading for sands. This is not supported by the FH tests in which considerable reduction in resistance occurred between initial loading to a given set of controlled deflections and that after a number of cycles.

(11) Excellent duplication of the initial series of PRH tests was obtained, using API static criteria as backbone curves in the SPASM input. However, the same backbone curves resulted in underestimating lateral resistance in the FH tests.

(12) There is a somewhat obscure anomaly in static criteria effects mentioned in (10) and (11), above. However, for cyclic loading there was remarkably good agreement throughout the tests between predictions based on API criteria for cyclic loading and the corresponding measured data. This included both load-deflection characteristics at the pile head and the bending moment curves.

8.2 Recommendations

The SPASM program appears to be quite adequate for analyzing the Seal Beach test results. The SPASM procedure can be adopted for lateral pile design under static or quasi-static loading. Lateral pile design for restrained-head cases is often dominated by the boundary restraint assigned in the analysis. Therefore, careful attention should be directed to proper computer modelling of the pile-structure connection (boundary condition).

The strain gaging method worked very well for this study; a similar procedure can be applied to other field measurements of piles and structures.

The downward migration of soil and compaction along the pile under cyclic loading are physically complex. The effect should be recognized but is secondary in importance and no attempt at modeling is recommended in ordinary design.

The application of API cyclic p-y criteria to the layered soil system duplicated the load test results with reasonable success. This procedure is therefore recommended for lateral pile design in layered soils.

Since the observed pile behavior was dominated by the sand layers, it may also be recommended that the API method continue to be used for quasi-static analysis for design of piles under cyclic loading conditions. Static loading predictions by API methods are not much different from the cyclic, but the observed results suggest that real static soil reactions may be somewhat greater than predicted.

9.0 REFERENCES

(1) American Petroleum Institute (API), (1982), "API Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms," API RP 2A, Thirteenth Edition, January 1982.

(2) Barton, Y. O., W. D. Liam Finn, R. H. G. Parry, and I. Towhata, (1983), "Lateral Pile Response and p-y Curves from Centrifuge Tests," Proceedings, 15th Annual Offshore Technology Conference, Houston, Texas, May 1983.

(3) Bogard, D., and H. Matlock (1977), "A Computer Program for the Analysis of Beam-Columns Under Static Axial and Lateral Loads, "Proceedings, 9th Annual Offshore Technology Conference, OTC 2953, Houston, Texas, May 1977.

(4) Bogard, D., and H. Matlock (1980), "Simplified Calculation of p-y Curves for Laterally Loaded Piles in Sand," An in-house technical paper, April 1980.

(5) Boutwell, Jr., G. P. (1968), "On the Yield Behavior of Cohesionless Materials," a Ph.D. dissertation, submitted to Duke University, School of Engineering, 1968.

(6) Chan, J. H. C., and H. Matlock (1973), "A Discrete-Element Method for Transverse Vibrations of Beam-Columns Resting on Linearly Elastic or Inelastic Supports," Preprints, Offshore Technology Conference, Houston, Texas, OTC 1841, May 1973.

(7) Cox, W. R., L. C. Reese, and B. R. Grubbs (1974), "Field Testing of Laterally Loaded Piles in Sand," Proceedings, 6th Annual Offshore Technology Conference, OTC 2079, Houston, Texas, May 1974.

(8) Ertec (1981), "Full-Scale Pile Vibration Tests," A report to The National Science Foundation, submitted by The Earth Technology Corporation and The California Institute of Technology, December 1981.

(9) Matlock, H., and E. A. Ripperger (1957), "Lateral Loading of an Instrumented Pile with Soil Conditions Varied," A report to Shell Oil Company, Houston, Texas, June 1957.

(10) Matlock, H., and E. A. Ripperger (1958), "Measurement of Soil Pressure on a Laterally Loaded Pile," Proceedings, American Society for Testing Materials, Vol. 58, Boston, Mass., 1958.

(11) Matlock, H. (1970), "Correlations for Design of Laterally Loaded Piles in Soft Clay, "Preprints, 2nd Annual Offshore Technology Conference, OTC 1204, Houston, Texas, May 1970.

(12) Matlock, H., S. H. C. Foo, C. F. Tsai, and I. Lam (1979), "SPASM 8: A Dynamic Beam-Column Program for Seismic Pile Analysis with Support Motion," a report prepared by Fugro, Inc., Long Beach, California, January 1979.

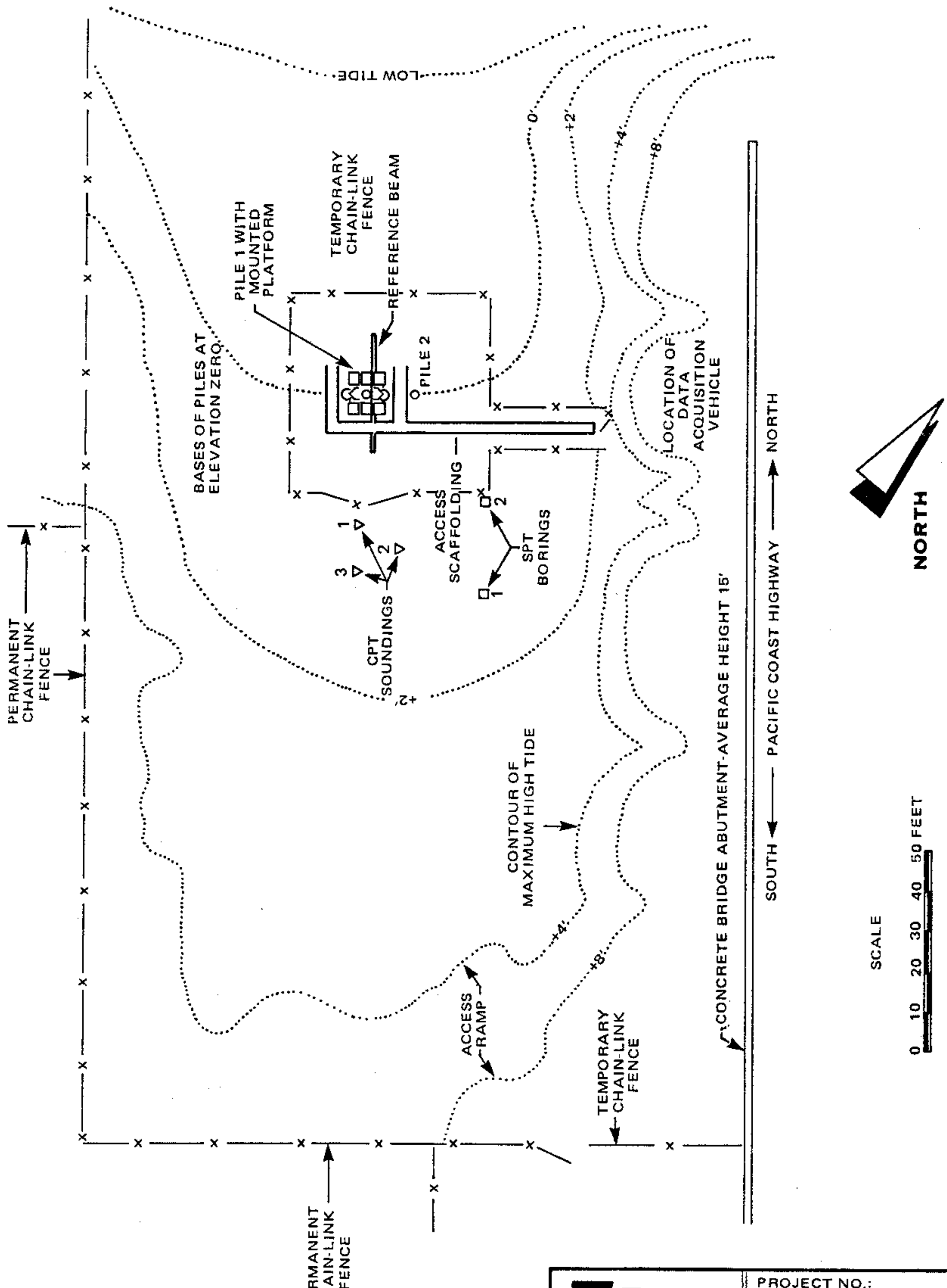
(13) Mitchell, J. K., and T. A. Lunne (1978), "Cone Resistance as Measure of Sand Strength," Journal of the Geotechnical Engineering Division, ASCE, No. G77, July 1978.

(14) Reese, L. C., W. R. Cox, and F. D. Koop (1974), "Analysis of Laterally Loaded Piles in Sand," Proceedings, 6th Annual Offshore Technology Conference, OTC 2080, Houston, Texas, May 1974.

(15) Scott, R. F., C. F. Tsai, D. Steussy, and J. M. Ting (1982), "Full-scale Dynamic Lateral Pile Tests," Proceedings, 14th Annual Offshore Technology Conference, OTC 4203, Houston, Texas, May 1982.

APPENDIX

LOCATIONS AND RESULTS OF PREVIOUS NSF SITE INVESTIGATION PROGRAM



Compiled by ______ / Drawn by ______ / Checked by ______ / Approved by ______

Ertec *The Earth Technology Corporation*

PROJECT NO.: 82-205

SEAL BEACH

PLAN VIEW OF SITE, SHOWING LOCATIONS OF PILES, SPT BOREHOLES, AND CPT SOUNDINGS (AFTER NSF REPORT, 1981)

8-83

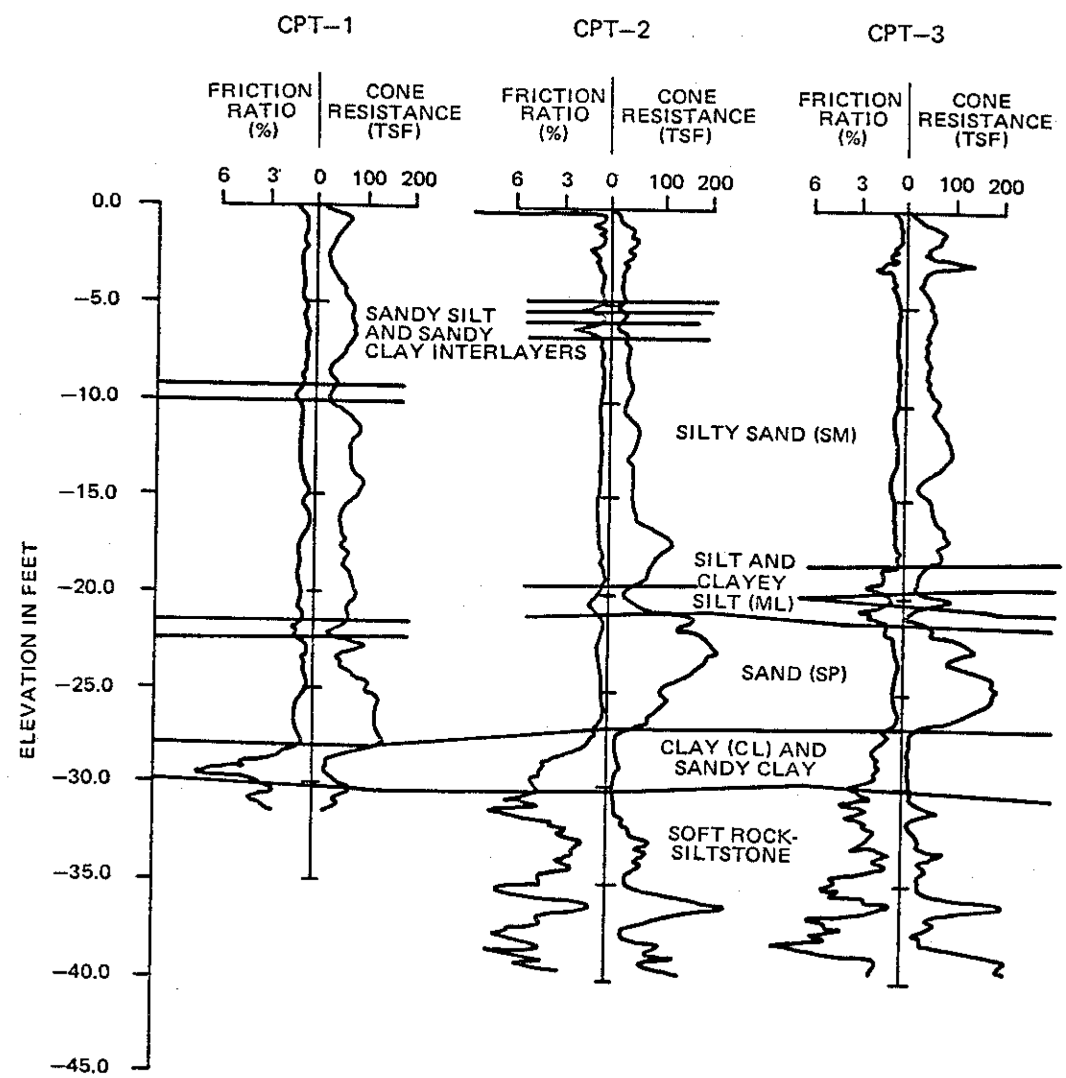
FIGURE A-1

Approved by ______ / ______

Checked by ______ / ______

Drawn by ______ / ______

Compiled by ______



LOCATIONS OF SOUNDINGS ON FIGURE A-1

Ertec The Earth Technology Corporation	PROJECT NO.: 82-205 SEAL BEACH

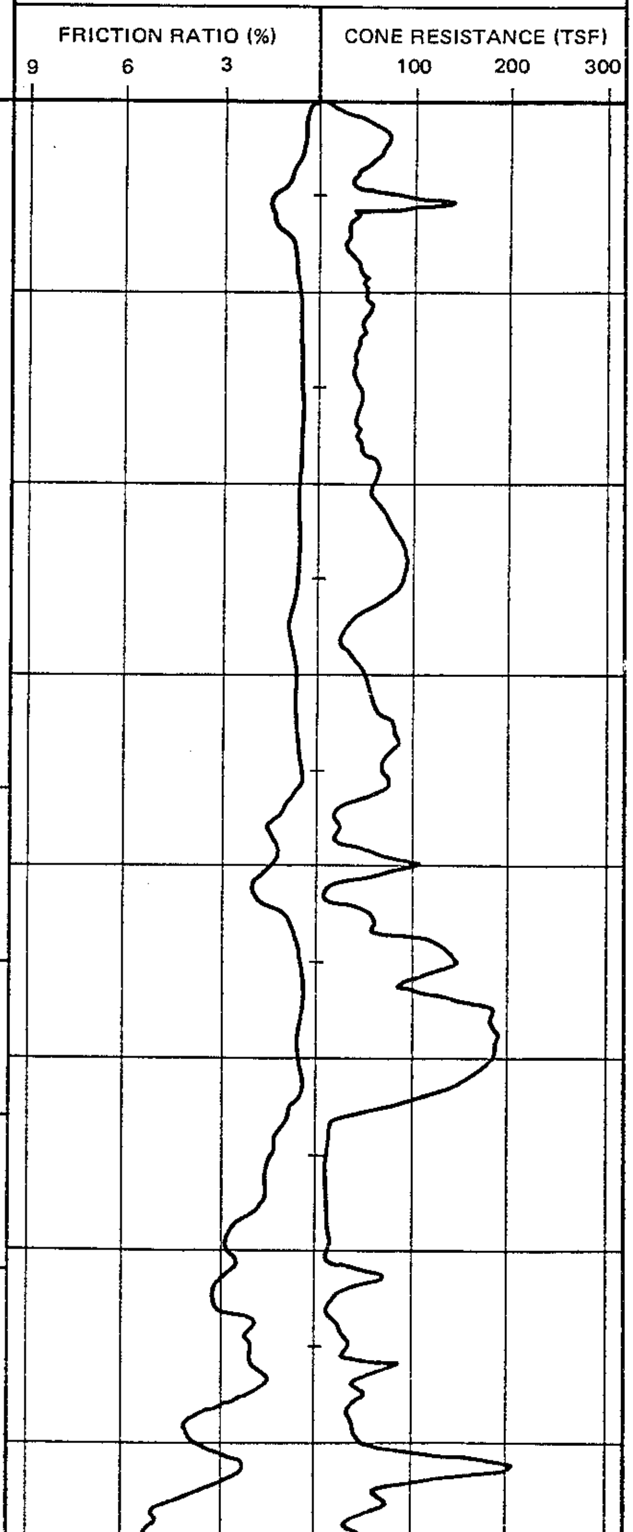
SEAL BEACH SITE CPT PROFILE
(AFTER NSF REPORT, 1981)

8-83

FIGURE A-2

DEPTH (FT)	SYMBOL	SAMPLE TYPE	BLOWS/FT*	SOIL DESCRIPTION
		S	13	DARK GRAY, FINE SILTY SAND WITH SCATTERED SILT AND SMALL SHELLS, LOOSE TO MEDIUM DENSE
		S	16	
5				
		NR	12	
		NR	13	
10		S	13	
		S	10	
		S	17	
		S	10	
15				
		S	14	
		S	9	
		S	11	DARK GRAY SILT WITH SOME SAND AND CLAY, MANY SMALL SHELLS, MEDIUM DENSE
20		S	10	
		S	20	
		S	25	DARK GRAY, MEDIUM TO FINE SAND, WITH SOME SILT, VERY SHELLY, DENSE
25		S	21	
		S	5	
		S	7	DARK GRAY, FIRM CLAYEY SILT
		S	14	
30				
		S	10	GRAY-GREEN FRAGMENTED SILTSTONE AND STIFF SILT
		S	28	
		S	22	
35		S	31	
		NR	23	

TYPICAL CONE PENETROMETER PROFILE
(CPT-3)



ELEVATION: MEAN SEA LEVEL DATE DRILLED: 11-18-80 (SPT)
EQUIPMENT USED: FAILING 750 & CPT TRUCK
WATER LEVEL: 0'

S STANDARD SPILT SPOON SAMPLE (ASTM D1586)
NR NO RECOVERY
*AVERAGE VALUE OF BORINGS 1 & 2 (WHEN APPLICABLE)
*STANDARD 140 LB HAMMER DROPPED 30"

Ertec
The Earth Technology Corporation

PROJECT NO.: 82-205

SEAL BEACH

SEAL BEACH SITE SOIL PROFILE
(AFTER NSF REPORT, 1981)

8-83

FIGURE A-3

Approved by ______
Checked by ______
Drawn by ______
Compiled by ______